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Swelling Characterization of Natural and Chemically Stabilized Clays using Centrifuge Technology

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16. Abstract

Expansive clays, which are common throughout Texas, may cause severe damage to pavements and other lightweight structures. Methods of direct quantification of soil swelling potential may be time-consuming, while indirect methods based on correlations to soil index parameters (e.g., plasticity and grain size distribution) rely on relatively scant empirical data. Also, indirect methods may fail to capture the effects of site-specific conditions, presence of distinct clay minerals, and pore-water composition. Also, indirect methods may not be accurate to optimize the design of chemical stabilization techniques aimed at minimizing the swelling of natural clays. Furthermore, indirect methods may require performing several additional tests in addition to plasticity and grain size distribution, in order to accurately characterize the clay composition, organic content, and sulfate minerals present in the natural soil deposits. Hence, in engineering practice, the design of a chemical stabilization program may involve measurement of a number of parameters except the most important one: the actual soil swelling. This is mainly because of the prohibitively long duration of soil swelling quantification using conventional techniques. The purpose of this report is to develop an accurate, yet expeditious approach to measure directly the swelling of expansive soils either in their native state, or after modification with chemical stabilizers. This is achieved by using centrifuge techniques, which have proven to be a reliable and swift approach to quantify clay swelling. Focus of this report is on the use of hydrated lime as a chemical stabilizer, although the procedures and findings discussed herein are expected to be also applicable to other stabilization techniques.

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Table of Contents

List of Tables

List of Figures

Chapter 1. Introduction and Organization

1.1 Introduction

Expansive clays are common in East, North, and Central Texas, much of the central US, and many other regions worldwide. These clays are responsible for significant damage to pavements and other lightweight structures, with an estimated annual damage to US infrastructure that exceeds $15 billion (Jones and Jefferson, 2012).

Expansive clays undergo significant volume changes in response to changes in moisture content—exhibited either as swelling upon wetting or shrinkage upon drying. Natural seasonal moisture fluctuations result in shrink-swell cycles in these materials within a certain depth below the ground surface, typically known as the “active zone.” When a pavement or other lightly loaded structure is founded upon these materials, the cycles in shrinkage and swelling may lead to flexure-induced distress, particularly towards the edges of the structure. In pavements, this tends to manifest itself in the form of longitudinal cracks, as repeated vertical movements of the pavement shoulders lead to fatigue cracking in the asphaltic layer (Zornberg and Gupta, 2009).

Other distress problems tend also to correlate with the expansiveness of pavement subgrades, including heaving, rutting, and general increase in roughness, all of which compromise the integrity of the asphaltic layer as a moisture barrier, exacerbating moisture fluctuations within and beneath the pavement structure.

Additionally, most of the Texas population resides in the vicinity of some of the most extensive deposits of expansive clay (e.g., in the major population centers surrounding Dallas, Fort Worth, Austin, San Antonio, and Houston) as shown in Figure 1.1.1. Consequently, existing and future infrastructure development face increasing costs aimed at retrofitting or mitigating the effects of expansive clay soils.

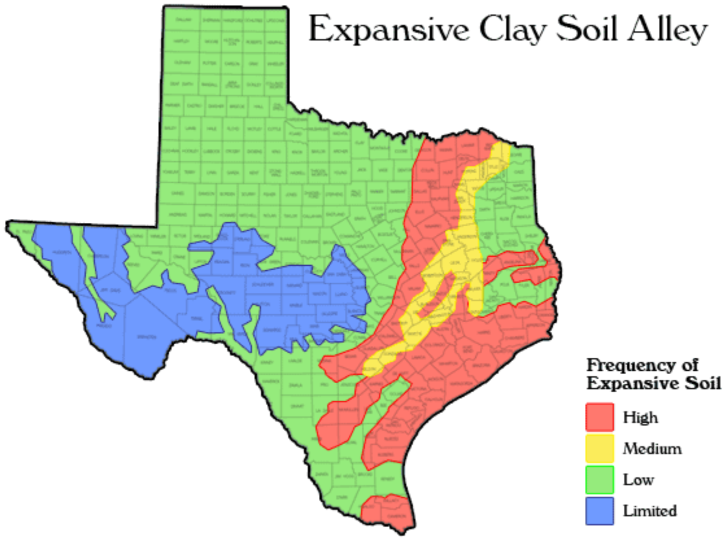


Figure 1.1.1: Expansive soil distribution in Texas (Tella Firma 2017).

Consistent with the prevalence of expansive clays worldwide, significant research efforts have been undertaken to understand and quantify the phenomenon of swelling in clay soils. Current methods to quantify the potential swelling of expansive clay soils include directly measuring the change in volume of a soil specimen after inundation (direct methods) or calculating a swelling magnitude based on index properties of the soil (indirect methods). The most common direct method is the free swell test, which is detailed in ASTM D4546. This test involves placing a compacted soil specimen in a consolidation frame and measuring the increase in height upon inundation with water, while maintaining a constant normal stress. While effective, these tests are time-consuming, with equipment requiring comparatively large space—often taking weeks or months to run to completion—and thus are often under-utilized.

Instead, indirect methods have been commonly used in practical applications. This includes the approach documented by McDowell (1955), which has been adopted by the Texas Department of Transportation (TxDOT) in method Tex-124-E. This method involves the determination of the plasticity index and initial moisture content of a given soil to estimate its potential swelling based on empirical correlations established using data from some expansive clay soils in Central Texas. Although this method is comparatively simple to implement, its empirical nature and reliance on a relatively limited swelling database may lead to inaccuracies and variability due to a number of reasons, including actual soil mineralogy and initial conditions.

To mitigate the detrimental effects of expansive soil on pavement structures, methods of soil stabilization have been extensively investigated, including techniques such as soil replacement, geosynthetic stabilization, and the treatment of subgrade soils with chemical stabilizers such as lime or Portland cement. The use of lime is especially common in Texas, partly because of the abundance of the raw material, limestone.

Lime treatment often has been adopted by TxDOT to reduce the potential swelling in roadway projects involving the presence of high plasticity subgrade soils. To determine the dosage of lime required for soil stabilization, a method known as the Eades-Grim Test has been used (ASTM D6276; Tex-121-E). This method involves mixing a series of soil slurries with increasing dosages of lime, and measuring the pH of the mixture. The lime dosage to be adopted for construction corresponds to the lowest lime dosage that leads to a pH of 12.4 in the mixture, at which point the pozzolanic reactions between clay particles and lime are expected to have fully occurred. This method is reasonably simple to perform and has been reported to provide insight on long-term soil *stabilization*; however, this method does not account for the effects of short-term soil *modification*, which are the most relevant effects to mitigate moisture-induced volume changes. Soil stabilization with lime involves cementation reactions that occur in the long term, but these reactions may require more lime than that necessary to decrease the soil's swell potential. Additionally, pavement design projects in which the shear stresses acting on the subgrade are relatively low may not necessarily benefit from the increase in strength due to pozzolanic reactions. In any case, the depth of treatment cannot be directly calculated using this approach, which is a critical aspect in the design of stabilized subgrade soils.

In light of the aforementioned shortcomings in the characterization process, a robust and expeditious method is still needed to properly quantify the swelling of clays and the impact on swelling of chemical stabilization. Accordingly, this implementation report focuses on refining the use of centrifuge testing to accelerate the collection of swelling information on expansive clays. Centrifuge technology does this by accelerating the moisture flow process during swelling tests on expansive clays, allowing specimens to reach full swelling in a comparatively short timeframe. The test can be performed directly on natural or on chemically-stabilized soils, ultimately allowing a rational evaluation of the benefits of chemical-treatment programs.

Several previous studies conducted during the development of this centrifuge method have demonstrated the accuracy, repeatability, and efficiency of the method. Additional data from conventional swelling tests in these studies have also confirmed their equivalence with the centrifuge method (Zornberg et al. 2008; Plaisted 2009; Kuhn 2010; Zornberg et al. 2013; Armstrong 2014).

After quantifying the magnitude of swelling as a function of confining pressure, the experimental results can be used to predict the vertical heave at the ground surface that would result from changes in the moisture content of subgrade soils. In particular, the ground heave predicted due to swelling for the case in which soil reaches saturation, often known as potential vertical rise (PVR) has been adopted to establish pavement design criteria. While the benefits of generating experimental centrifuge test results is not limited to their use for PVR prediction, experimental results generated as part of this project are used within the PVR framework, as they provide a soil-specific and project-specific index value relevant to roadway design.

1.2 Objectives and Scope of the Project

One of the main objectives of this project was to refine a testing and analysis procedure suitable to determine the swell-stress curves needed to predict the PVR in sites characterized by the presence of expansive clay subgrades. An additional goal was to establish an approach suitable to determine the depth of treatment with hydrated lime that is required to decrease the PVR at a site from its original, pre-treatment value to a prescribed value established as a design criterion. The testing procedure adopted in this project to predict PVR uses centrifuge technology to directly measure swelling strains instead of their determination using empirical correlations, as currently adopted in Tex-124-E. In support of these objectives, a systematic sensitivity evaluation was also conducted on various aspects of the testing procedures to assess their effect on the swelling results.

A final objective of this project is to illustrate the proposed procedures by implementing them using actual TxDOT roadway sites, several of which have been identified as requiring rehabilitation.

1.3 Organization of Report

This report is organized as follows:

- Chapter 1discussed the relevance of the problems associated with expansive clays for transportation projects, and provides the background and objectives of this research implementation project.
- Chapter 2 provides the results from the testing program on several natural clay soils, and details the major findings from studies into the initial conditions of testing and evaluations of lime-treatment dosages.
- Chapter 3 reviews several of the sample preparation and testing protocols adopted herein for their impact upon swelling test results.
- Chapter 4 outlines the use of the centrifuge test method for practical design purposes.
- Chapter 5 provides a description of the activities, data collection, and findings regarding a number of actual roadway sites evaluated as part of the field-support effort within this project.
- Finally, the overall conclusions of the project are provided in Chapter 6.

Chapter 2. Centrifuge Testing Program

Soil samples were collected from a number of field sites in Texas both in order to evaluate the swelling behavior of a wide range of clay soils, and to allow an evaluation of a site-specific PVR at each of these sites. Characterization of the swelling behavior of selected soils is described in this chapter (Chapter 2), while a description and evaluation of the predicted PVR at every site can be found in Chapter 5.

A comprehensive testing program was performed using these selected soils in their natural (untreated) state to evaluate the effects of the initial conditions of testing, and to identify possible correlations between swell results and clay index parameters. A subsequent testing program was performed to determine the effects of lime-treatment at different dosages upon the clay swelling behavior.

Section 2.1 describes the general characterization of the different soils used in this evaluation. Section 2.2 provides an evaluation of relevant parameters affecting swelling in untreated soil specimens. Finally, Section 2.3 evaluates the additional relevant parameters affecting swelling in lime-treated soil specimens.

2.1. General Soil Characterization

In order to compare the swelling behavior of highly expansive soils in their natural and treated state, three soils were chosen for a baseline testing program involving extensive index testing and centrifuge swell testing.

The three soils used in this baseline testing program include:

- Clay samples from a high plasticity shale layer within the Eagle Ford formation, collected from a highway cut at Interstate Highway 35 (I-35) and Hester's Crossing in Round Rock, Texas.
- Clay samples from a high plasticity shale collected from a highway cut made in the Taylor Shale at the intersection of US 183 and Martin Luther King Jr. Blvd in Austin, Texas.
- Samples from a high plasticity clay from the A-horizon of a Houston Black Clay deposit, collected from a site at Loop 1604 and Graytown Rd, in San Antonio, Texas.

Additional soils considered within the broader testing program to establish basic material behaviors include:

- Samples from a wide range of clay soils at Farm-to-Market (FM) Road 2 in Grimes County, Texas, used to evaluate the impact of clay plasticity upon swelling

- Samples from clay soils with a relatively high fraction of coarse particles from FM 972 in Williamson County, Texas, used in the evaluation of the impact of soil binder content upon swelling.

Figure 2.1.1 shows a map of the locations from which soils were sampled as part of this testing program.

Figure 2.1.1: Sampling locations included in baseline soil series.

Table 2.1.1 shows the results of the characterization on the three high plasticity clays used as baseline soils.

Table 2.1.1: Characterization properties of baseline soils.

Soil	Source Location	Liquid Limit	Plasticity Index	% passing #200 Sieve	Clay Fraction (<0.002mm)	Standard Proctor Optimum Water Content	Standard Proctor Maximum Dry Unit Weight
		(%)	(%)	(%)	(%)	(%)	(pcf)
Eagle Ford	I-35 & Hester's Crossing	83-88	49-60	97	77	25	94
Houston Black A	Loop 1604 & Graytown Rd.	79	53	90	56	26.5	90
Taylor Clay	US 183 & MLK Jr. Blvd	71	51	93	50	23.5	98

Figure 2.1.2 shows the compaction curves obtained using Standard Proctor compaction tests (ASTM D698) on soil samples collected from the three baseline-series sites. The degree of saturation curves were generated considering a specific gravity of solids of 2.7.

Figure 2.1.3 presents the grain size distribution curves obtained for the three baseline soils included in Table 2.1.1. As seen in the data, these soils are all high plasticity clays with a significant fraction of clay-size particles. While the Houston Black soil is the most similar in silt content and plasticity to the Taylor Shale, the maximum dry density is more similar to the Eagle Ford clay, possibly because of the clay content.

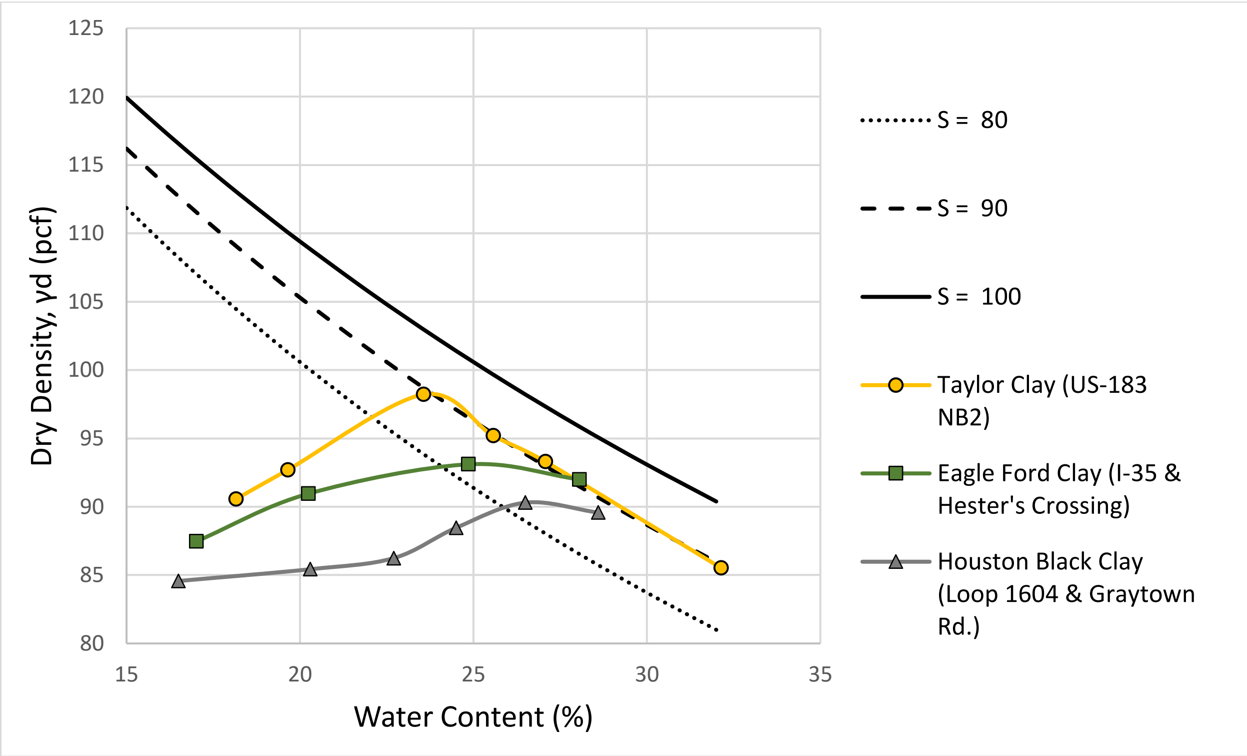


Figure 2.1.2: Standard Proctor compaction curves for baseline soils.

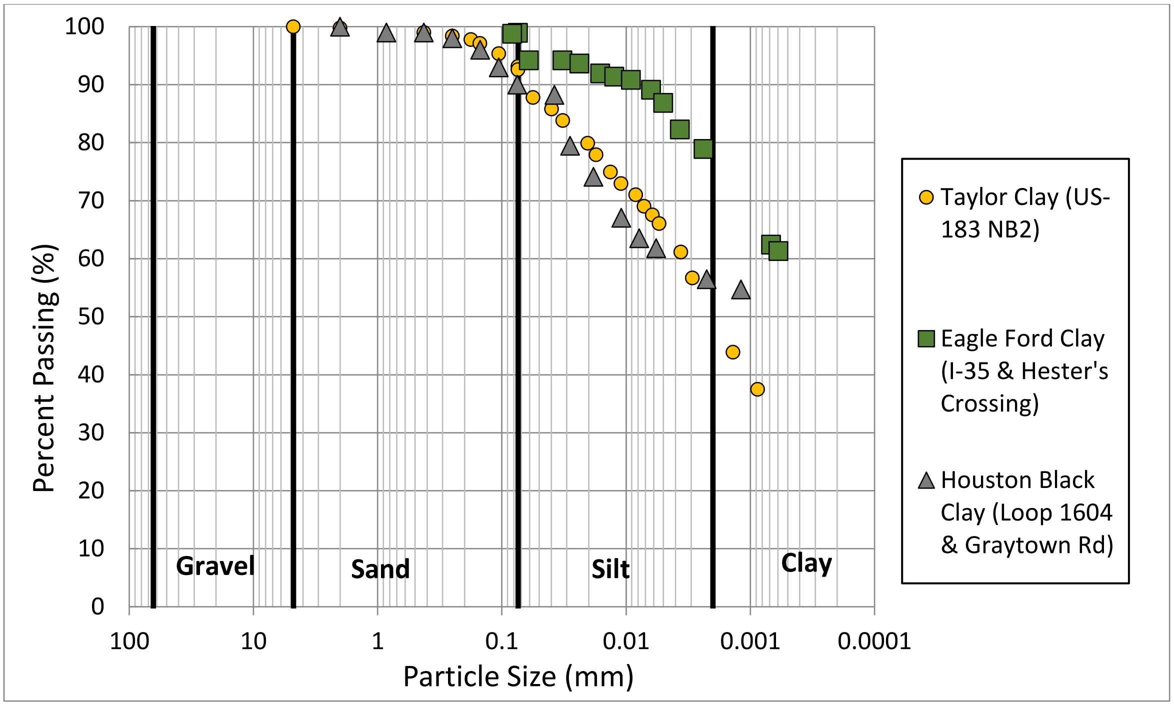


Figure 2.1.3: Grain size distributions for selected baseline soils.

2.2. Results from Texas Swell Tests on Natural Soils

2.2.1. Generic Swell-Stress Behavior: Effect of Vertical Effective Stress

A testing program was performed on the Eagle Ford Clay to demonstrate the relationship between the vertical effective stress and the swelling behavior of a highly plastic clay soil.

Soil samples from the Eagle Ford Clay were air dried, processed in a rock crusher, and moisture conditioned by hand to a precise target value. Specimens were prepared in accordance with the revised testing protocols for kneading compaction outlined in Appendix A: Testing Protocols for the Texas Swell Test.

The target initial conditions used to prepare the soil specimens and generate the centrifuge swell-stress curves for Eagle Ford Clay are shown in Table 2.2.1. These target initial soil conditions were chosen as 1 point dry of the optimum moisture content and 100% of the Standard Proctor maximum dry density.

Table 2.2.1: Compaction conditions for untreated Eagle Ford Clay swelling data.

	Moisture Content	Compaction Void Ratio	Dry Density [pcf]
Target	24%	0.820	94.0
Minimum Value	23.5%	0.797	92.5
Maximum Value	24.5%	0.861	99.3
Maximum Error from Target	± 2.1%	± 5.0%	± 5.0%

Figure 2.2.1 shows the swell-stress data from this testing series. The data show that this soil, when compacted to near Standard Proctor optimum conditions, can swell up to 16% under a load of 100 psf, and that the swell pressure, defined here as the load under which no swelling will occur upon wetting, is well above 3000 psf. Additionally, this data indicates that a simple log-linear fitting of the data is sufficient to describe the relationship between swelling and effective stress. This is particularly relevant in the stress range of 100 psf to 1000 psf, which is the most important range to PVR calculations. This detail in the modeling of the data will be explored further in Chapter 4, specifically as it relates to the incorporation of data from lime-treated samples into an analysis of the treated soil behavior.

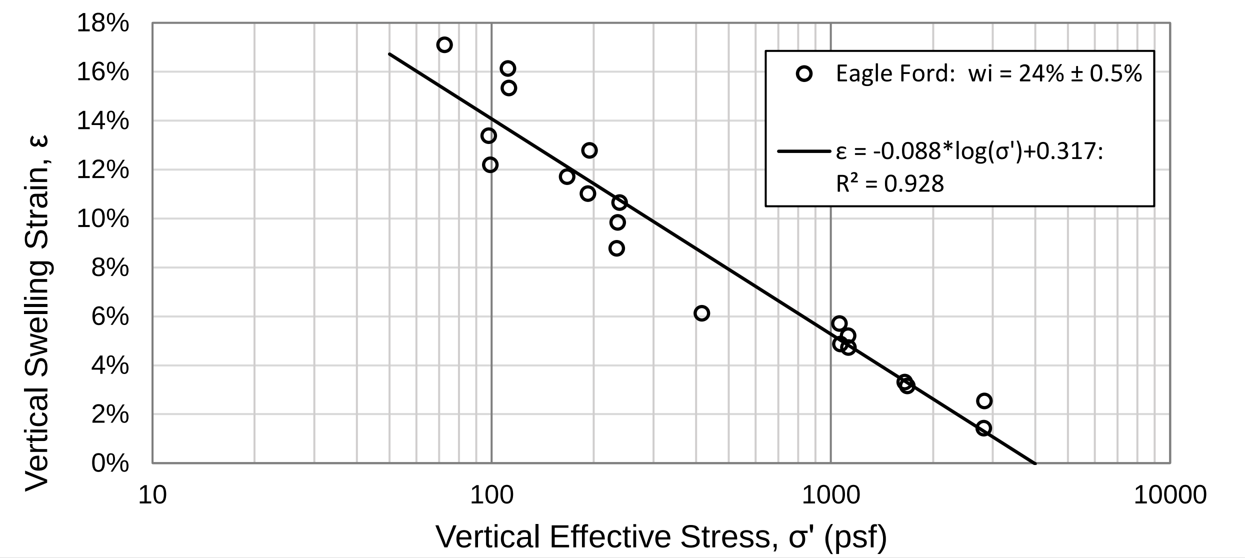


Figure 2.2.1: Swell-stress curves obtained in centrifuge test of natural Eagle Ford Clay.

2.2.2. Effect of Initial Density and Moisture

Because volume change in expansive clays takes place in response to moisture changes, the magnitude of swelling upon wetting is directly linked to the initial conditions of moisture content and dry density in the soil, prior to adding additional moisture.

A soil specimen with a comparatively high initial moisture content is expected to swell less than a specimen of the same soil with a lower initial moisture content prepared at the same initial dry density, because clay particles in a comparatively drier specimen will have a higher potential to adsorb water.

It is also expected that comparatively denser soils would have a higher potential to swell than looser soils for a given initial moisture content, simply because there are more active clay particles packed into the same volume (assuming no net-positive attractions occur between these particles with denser packing).

To quantify these expectations, parametric evaluations were conducted using the initial moisture content and dry density as variables, observing their impact upon the swelling. Separate testing series were conducted using two of the baseline soils: the Eagle Ford Clay and the Taylor Clay.

The first test series involved testing of the Eagle Ford clay, which is known for its comparably high potential for swelling. In this series, one set of specimens was prepared with a constant dry density, but having variable moisture content, while a second set of specimens was prepared with identical moisture content, but having variable dry density. Each set of specimens was tested at an identical stress to allow direct comparison among the results.

Figure 2.2.2 shows the trend of decreasing swell with increasing initial moisture content for untreated Eagle Ford clay tested at a target effective stress of 95 psf (actual stress values during testing ranged from 90 psf to 100 psf). A well-defined linear trend can be observed. Fitting the data to a straight line results in a slope of -1.1, which shows that for Eagle Ford clay, changes in initial moisture content have a nearly 1-to-1 correlation with changes in swelling magnitude (at the target effective stress of 95 psf).

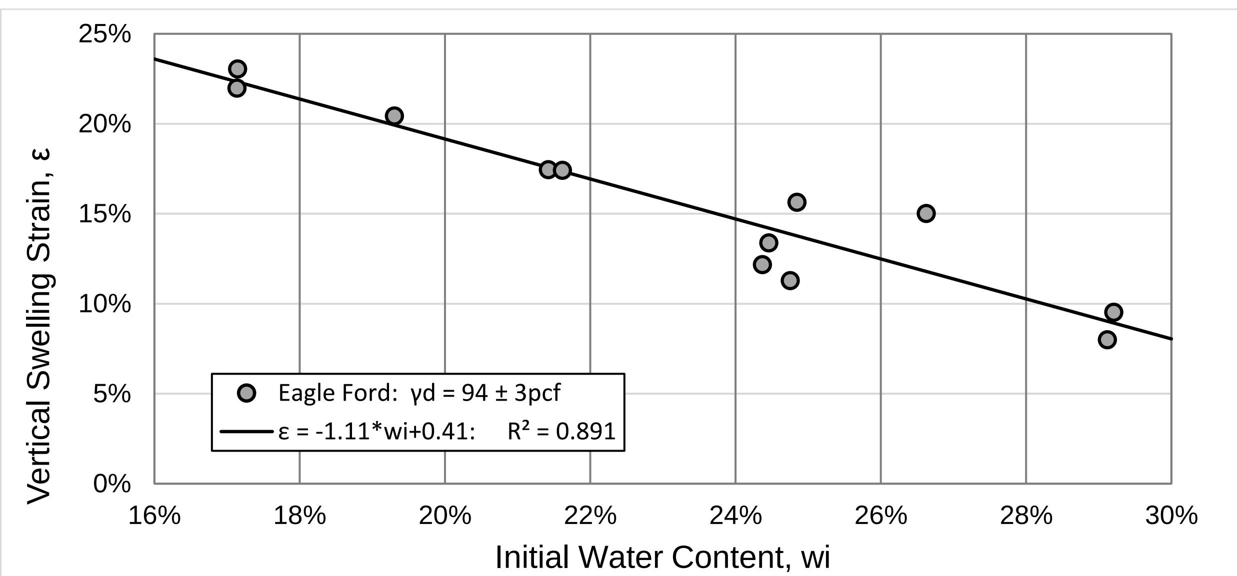


Figure 2.2.2: Impact on swell results of initial moisture content for untreated Eagle Ford Clay at 95 psf.

Figure 2.2.3 shows the variation of swell with compaction dry density for untreated Eagle Ford clay tested at an effective stress of approximately 285 psf (stress values vary from 280 psf to 298 psf). Also in this case, a roughly linear trend is evident between swelling magnitude and increasing compaction dry density. When the data is fit to a straight line, a correlation can be found with an increase in swelling of approximately 1% for an increase in dry density of 3.2 pcf (at this effective stress).

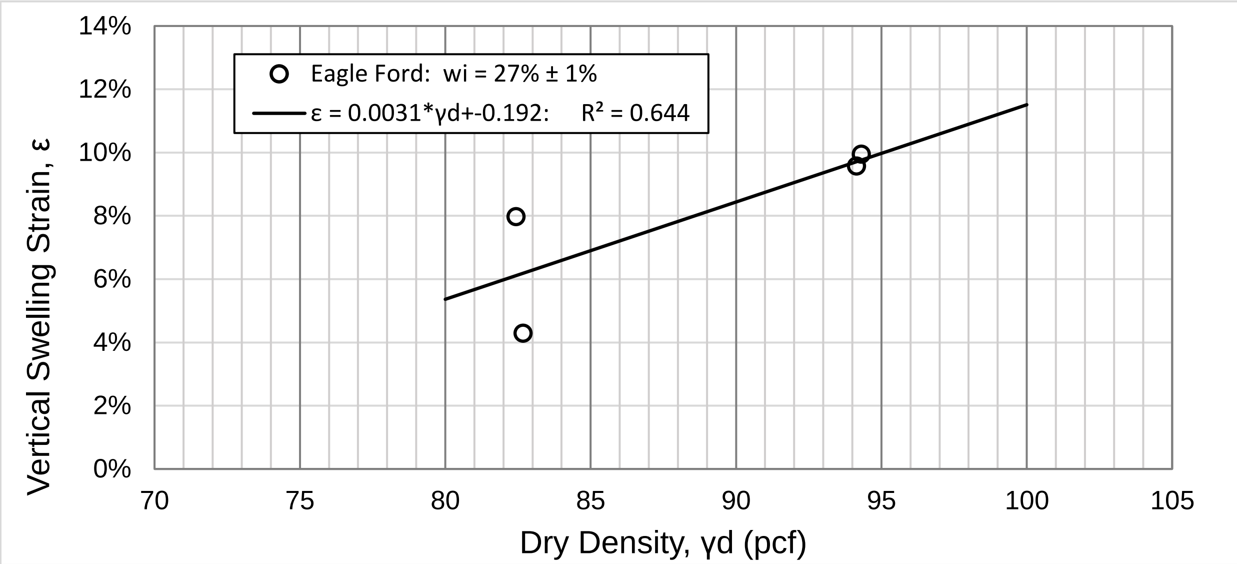


Figure 2.2.3: Variation of swell with dry density for untreated Eagle Ford Clay at 285 psf.

One outcome of this testing series is the identification of acceptable limits of specimen variation in order to have comparable outcomes in the swelling data.

Based on this dataset, for swelling results to be adequately comparable (that is, to be within ±1% strain of the expected value), the actual moisture content of a specimen should be within ±1% of the target moisture content, and the initial dry density should be within ±3.2 pcf of the target dry density value.

Because the tests in this testing series were conducted at comparatively low effective stresses and on specimens of a particularly highly expansive soil, it is expected that this recommended precision in target initial conditions will lead to comparatively lower variability in test series conducted at comparatively higher pressures and in less-expansive clays.

In order to validate the suitability of these recommended limits, a second test series was performed using a moderately expansive clay soil (Taylor Clay) collected at the intersection of US 183 and Martin Luther King Jr. Boulevard in Travis County, Texas. Characterization of this site is described in more detail in Section 5.8. Soil samples tested as part of this series were initially homogenized, since the natural soil deposit itself was highly stratified.

In this second test series, specimens were prepared into two groups. The first group comprises specimens prepared to comparatively dense initial conditions, and achieving a degree of saturation greater than 50%, while the second group comprises specimens prepared to comparatively loose and dry initial conditions, and achieving a degree of saturation less than 50%. It is expected that the samples in the first group are more representative of soil states that are more likely to occur under actual field conditions. This expectation additionally has been incorporated into the revised testing protocols by prescribing initial density and water content values resulting in a degree of saturation of 85%, as detailed in Appendix A.

Figure 2.2.4 shows the initial moisture and dry density of the specimens from each of the two groups. In addition, this figure shows the final dry density and moisture content after swelling for each specimen. The results in Figure 2.2.4 show that the swelling results from tests conducted on specimens prepared comparatively loose and dry resulted in comparatively higher scatter in the final density and water content, than from tests conducted on specimens prepared comparatively dense and wet. The vertical effective stresses in this testing program ranged from 100 to 400 psf.

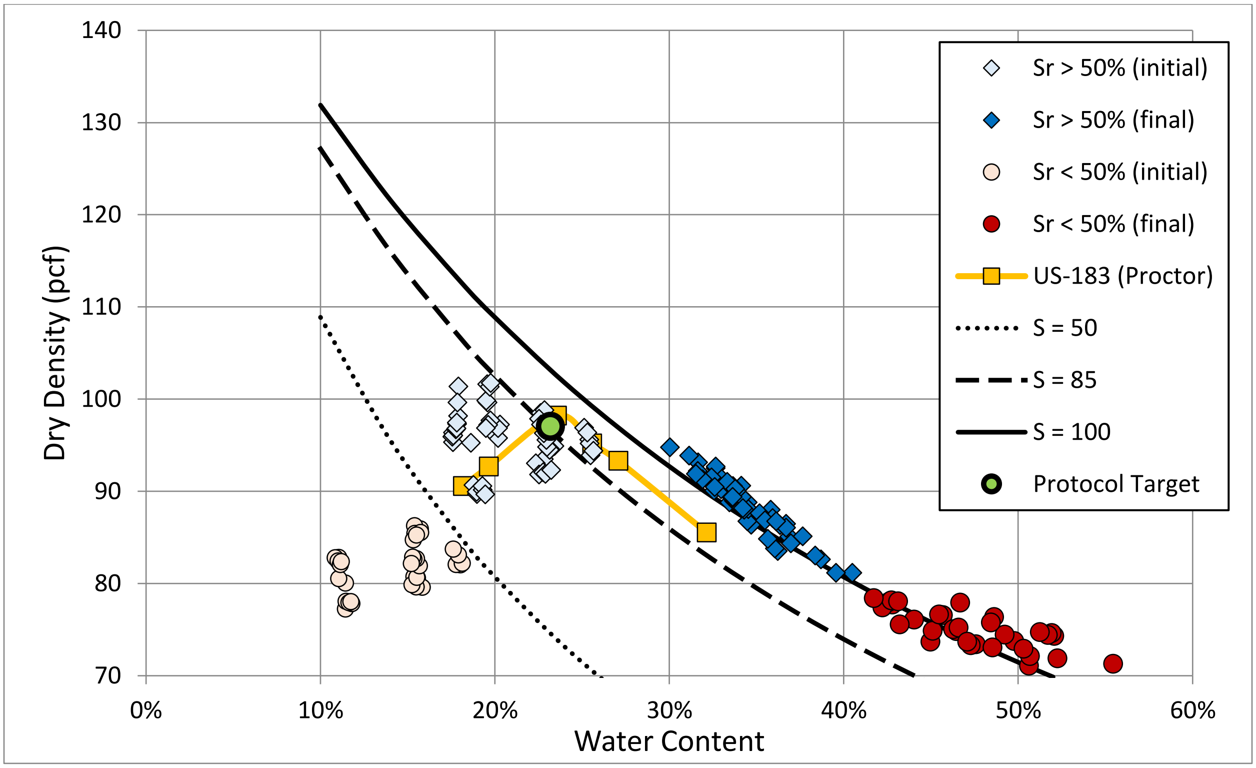


Figure 2.2.4: Dry unit weight and water content of specimens as obtained before and after swelling of Taylor Clay specimens.

Results obtained from this test series were grouped by vertical effective stress, and a combined evaluation of the contributions from dry density and water content was made using only the data obtained from the population with an initial degree of saturation greater than 50%. Data from the population with a degree of saturation less than 50% exhibited a significant amount of scatter as shown in Figure 2.2.5, and exhibited some signs of collapse along with swelling during testing. This data was subsequently excluded from consideration, except for observing that these soil conditions would not be expected in most field situations.

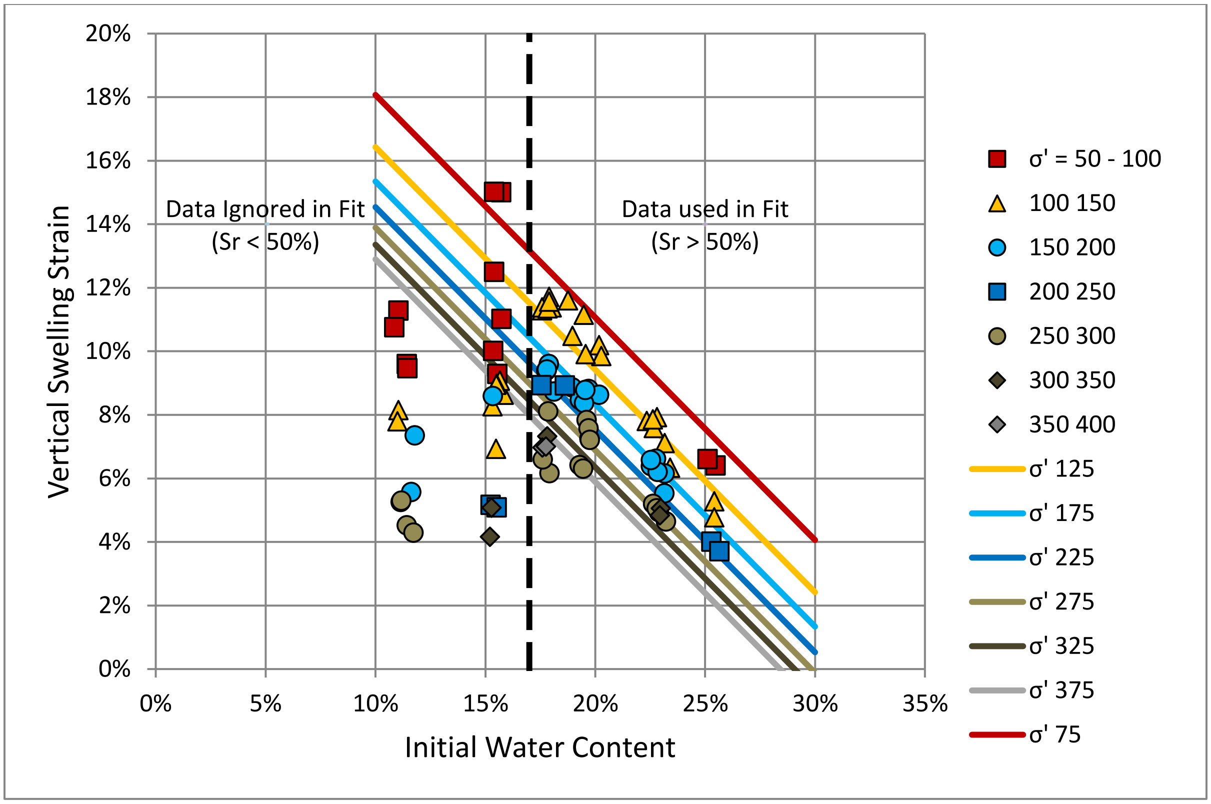


Figure 2.2.5: Swell for varying initial moisture content, also showing contours of effective stresses.

Evaluation of the test results was performed using a multiple regression considering the effective stresses (in decimal logarithm scale), initial moisture content, and initial dry density. Table 2.2.2 shows the parameters obtained from this analysis. It should be noted that the impact of the initial dry density on this data is minor. Instead, the impact of the initial moisture content is significant (a change in initial moisture of 1% results in a decrease in swelling of 0.7%). This relationship between swelling and initial moisture content holds true over the entire stress range of 100 to 400 psf used in this analysis, and for the data with a degree of saturation greater than 50%. Data from the population with a degree of saturation less than 50% likely exhibited some collapse along with a minor amount of swelling, so the relationship between the initial conditions and the swelling in these tests is more complex.

Table 2.2.2: Parameters returned from multiple regression on data with Sr > 50%.

dε/d(Log(Stress))	dε/d(w_i)	dε/d(γ_d) [in/in/pcf]	ε intercept
-0.074	**-0.700**	**0.0003**	**0.361**

Despite the minor influence of dry density upon the dataset of swell tests, the results were adjusted to a target initial dry density of 95 pcf to allow a more direct comparison of the effects of initial moisture. The adjustment is based on the multiple regression analysis of the data, and is obtained as follows:

$$\varepsilon_{adjusted} = \left(\gamma_{d_{target}} - \gamma_{d_{measured}}\right) * \left(\frac{d\epsilon}{d\gamma_d}\right) + \epsilon_{measured} \qquad (2.1)$$

where $\frac{d\epsilon}{d\gamma_d}$ is the relationship between swelling and initial dry density, determined from the global best-fit multiple regression. It should be noted that the maximum adjustment for this dataset is only on the order of 0.15% swelling strain. Figure 2.2.6 shows the initial dry density-adjusted data along with the model contours of moisture content at a density of 95 pcf. These results show that an increase in the initial moisture leads to a clear decrease in swell. However, the information gathered for both Taylor Clay and Eagle Ford Clay confirm that adjustments for initial moisture content is soil-type dependent, and is related to the overall expansiveness of the soil to begin with. Hence, any moisture-correction to the swelling data should be approached with care unless a significant dataset exists to support such a correction.

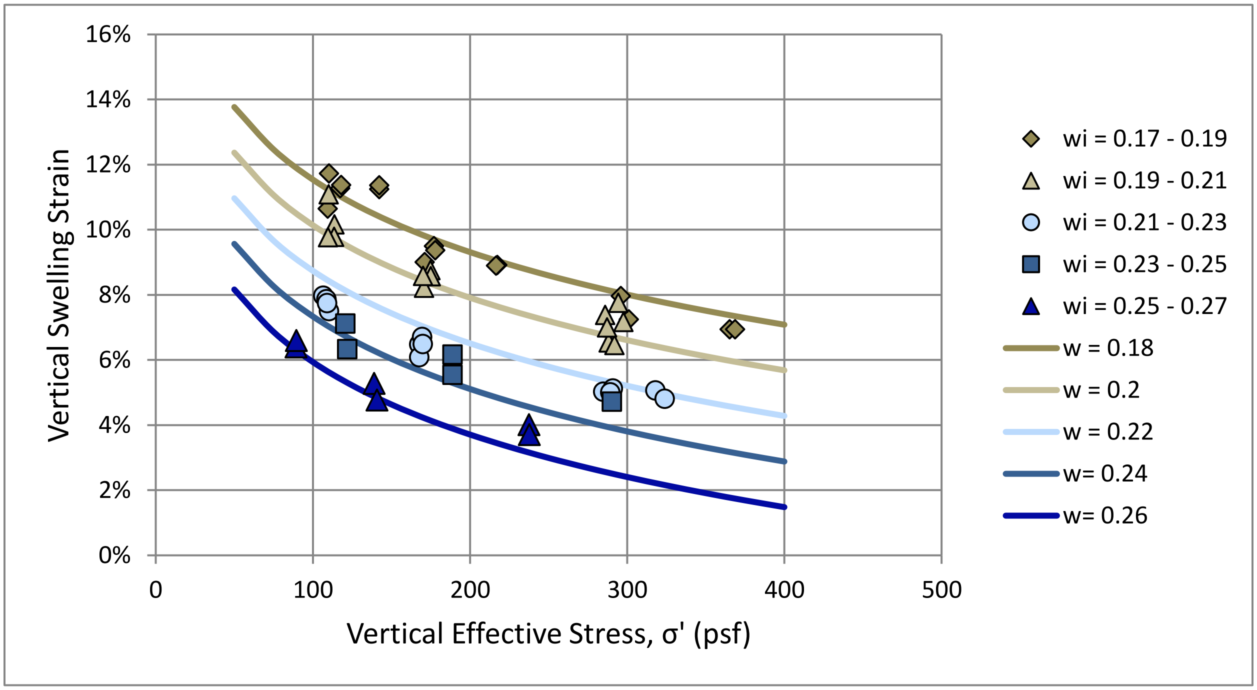


Figure 2.2.6: Swell-stress data for dense soil specimens of Taylor Clay, showing moisture contours from multiple-regression fit.

2.2.3. Relation between Swelling Results and Plasticity

The Atterberg Limits (Liquid and Plastic Limits) of soils are index properties that describe the relative amount of water the fine portion of a soil can hold at certain physical transitions in the soil behavior. They have been often used as simple surrogates for the much harder to obtain quantification of the mineralogy of a given soil sample. In fact, methodologies such as the Tex-124-E rely heavily on early correlations with these physical limits with the expectation that they would be a reasonable predictor of the swelling behavior of untreated soils.

Moisture-adjusted undisturbed soil samples from FM 2 were used to evaluate the correlation between the Liquid Limit of the clay and the measured swelling. Samples were collected from the top 10 feet below the ground surface along FM 2 as described in Section 5.3. The approximate field density and in-situ moisture contents were obtained by measuring the mass and dimensions of carefully-trimmed sections of Shelby tubes collected from the site. Figure 2.2.7 shows the average in-situ density and moisture content values obtained from the field.

These samples, collected from the top 10 feet, show an approximate trend in the in-situ water content vs. dry unit weight for a given liquid limit. At the time of sampling, materials with comparatively higher plasticity tended to be wetter and less dense, while soils with comparatively lower plasticity tended to be drier and denser.

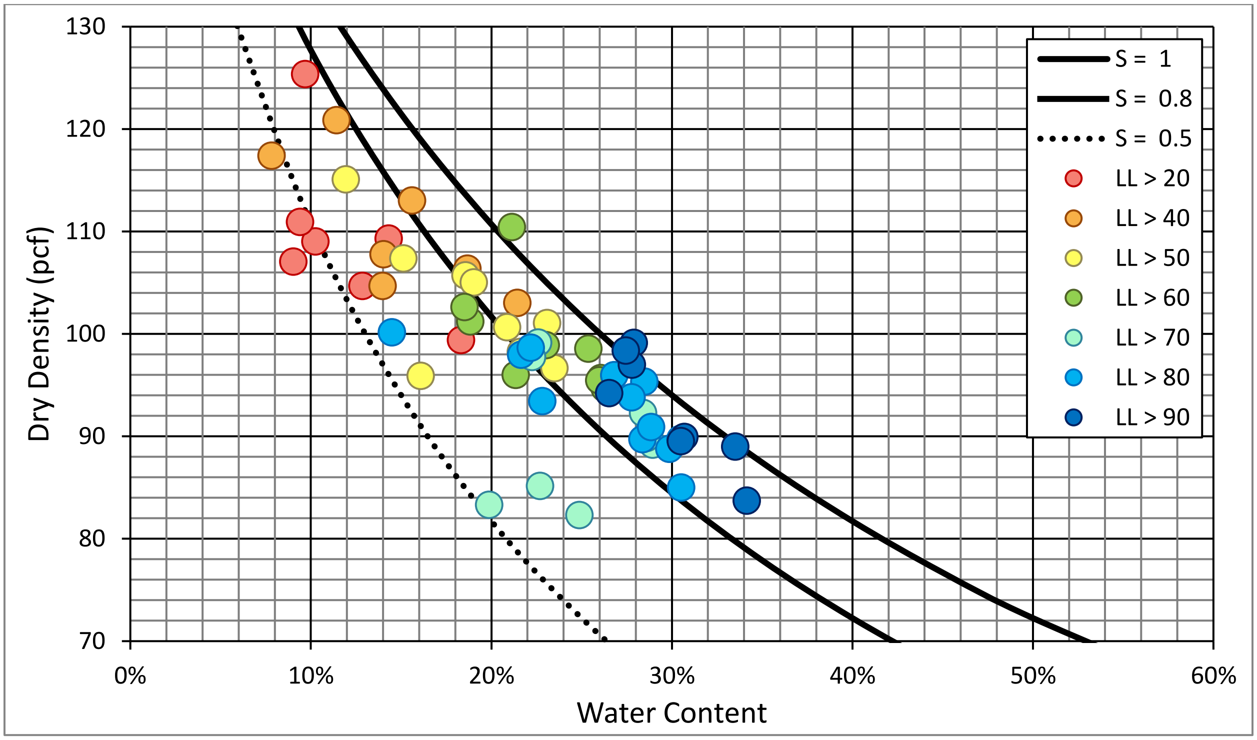


Figure 2.2.7: In-situ dry density and water content from undisturbed clay samples collected at the FM 2 site, grouped by liquid limit.

After evaluating the in-situ conditions, the Shelby tube samples were then sectioned and allowed to dry out slowly in an environmental chamber. For consistency, the target initial moisture content was chosen as the 'dry' moisture condition prescribed by Tx-124-E, as follows:

$$w_{dry}(\%) = 0.2 * LL + 9\% \qquad (2.2)$$

After reaching the target initial moisture content, the samples were trimmed into cutting rings for subsequent centrifuge testing. The initial moisture content for each centrifuge specimen is shown in Figure 2.2.8. It can be observed that the actual water content achieved is highly variable, but that the target condition was achieved on average. Figure 2.2.9 shows the range of initial conditions grouped by liquid limit with contours of the TxDOT dry equation plotted as vertical lines for reference. It should be noted that, in comparison to the previous parametric evaluation on compacted soils, only very few of the moisture-adjusted specimens from the field are in the range of S_r <50% and γ_d < 90 pcf. Additionally, the dry density values reported in this plot are expected to be lower bounds for the in-situ density, since minor voids may occur in the edges of the specimen during the trimming process into the cutting rings, reducing the measured mass value.

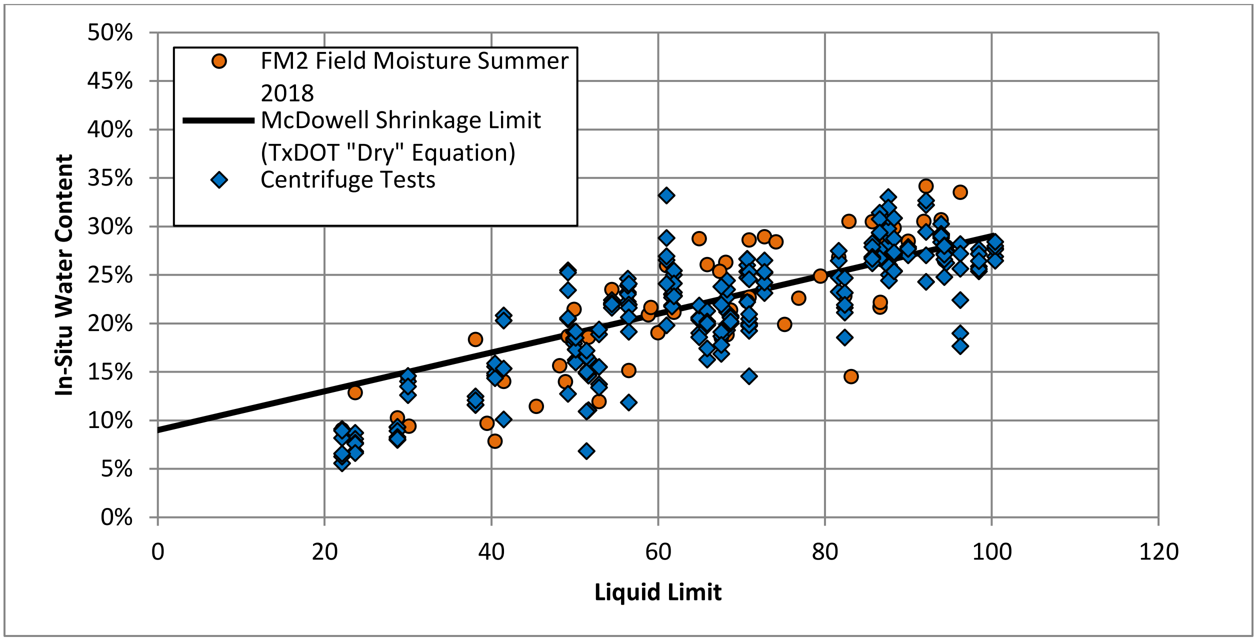


Figure 2.2.8: Field in-situ water content compared with actual initial moisture content of centrifuge specimens (target moisture content conditions also shown as reference).

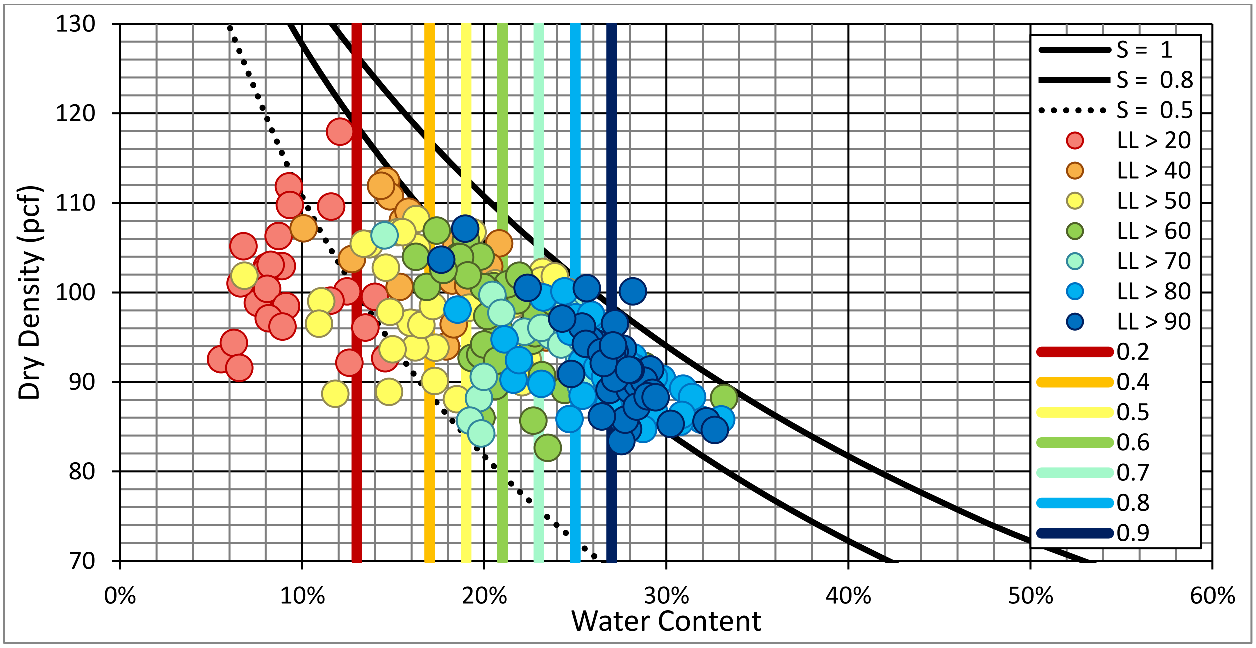


Figure 2.2.9: Initial conditions of centrifuge specimens grouped by liquid limit (target conditions also shown as vertical lines for reference).

Figure 2.2.10 and Figure 2.2.11 show the dry density after swelling, and after allowing each specimen to rebound under a very small load, respectively. A significant trend is observed in the higher plasticity soils swelling and rebounding more, while the lower plasticity soils tend to swell less, collapsing upon wetting in some cases.

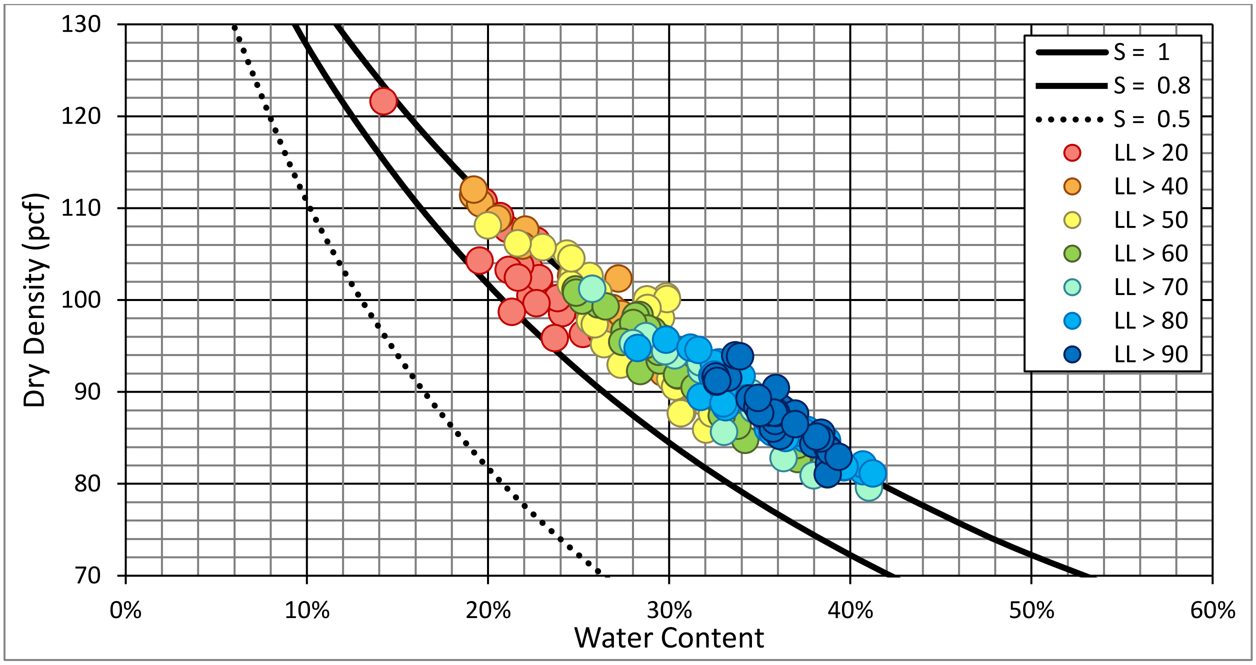


Figure 2.2.10: Final conditions (dry density and moisture content) at the end of primary swelling, grouped by liquid limit.

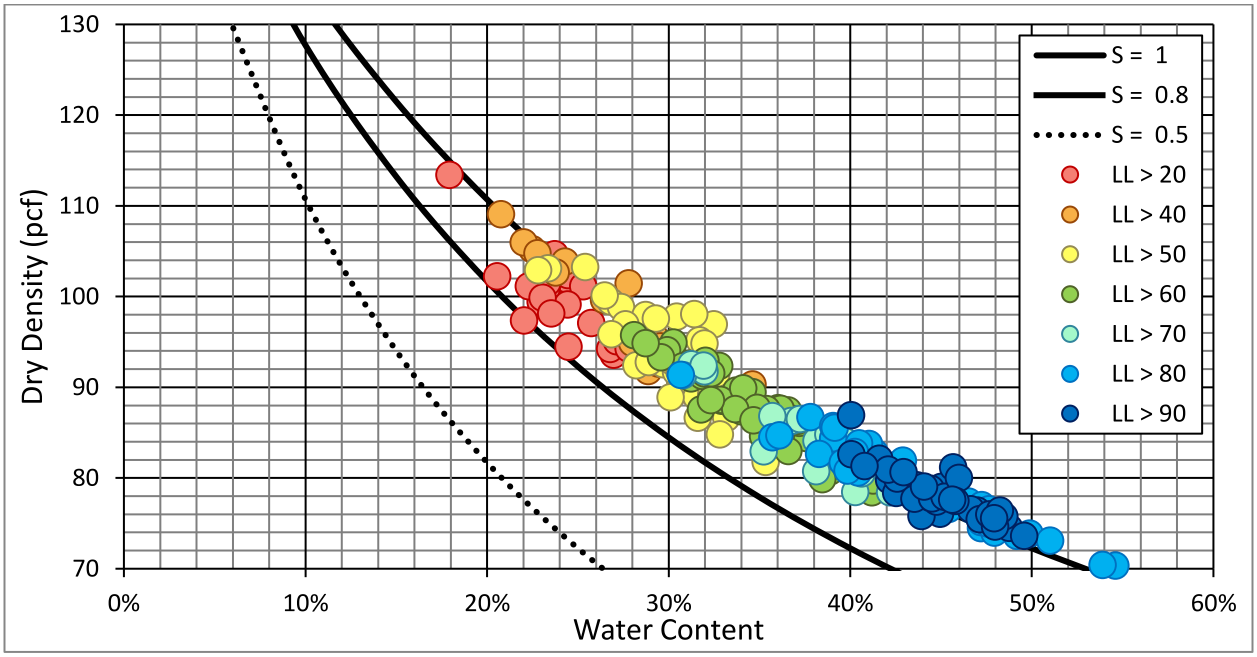


Figure 2.2.11: Densities at the end of rebound to a very small stress, grouped by liquid limit.

After dismantling each test, several test specimens were allowed to slowly dry out under atmospheric conditions for several days to limit cracking and warping of the soil, and were subsequently placed in an oven at 110 °C to complete the drying process. After oven drying, the volume was measured using dial calipers, to allow an estimate for the dry density after complete shrinkage. Figure 2.2.12 shows the dry density measured after oven-drying on selected specimens. It can be seen that all of these fine-grained soils with liquid limit greater than 50 tend to shrink to an average density of 116 pcf under minimal overburden during the drying process. Figure 2.2.13 shows swell-rebound- drying shrinkage paths followed by selected centrifuge specimens (including intermediate air-drying stages under minimal overburden) for soils of a wide range of Liquid Limit. These paths show that soils of comparatively high Liquid Limit tend to exhibit swelling and shrinkage over a very wide range of possible densities, while low Liquid Limit soils tend not to change volume significantly. Additionally, high Liquid Limit soils starting in the 'dry' condition still exhibit a significant amount of shrinkage after being dried below the 'dry' condition.

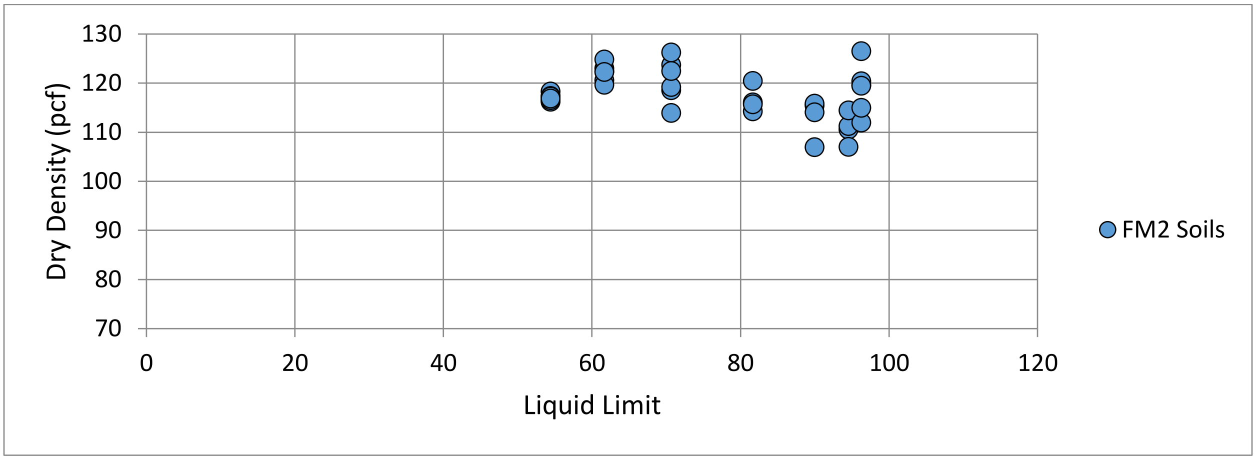


Figure 2.2.12: Density of clay specimens after oven drying, grouped by liquid limit.

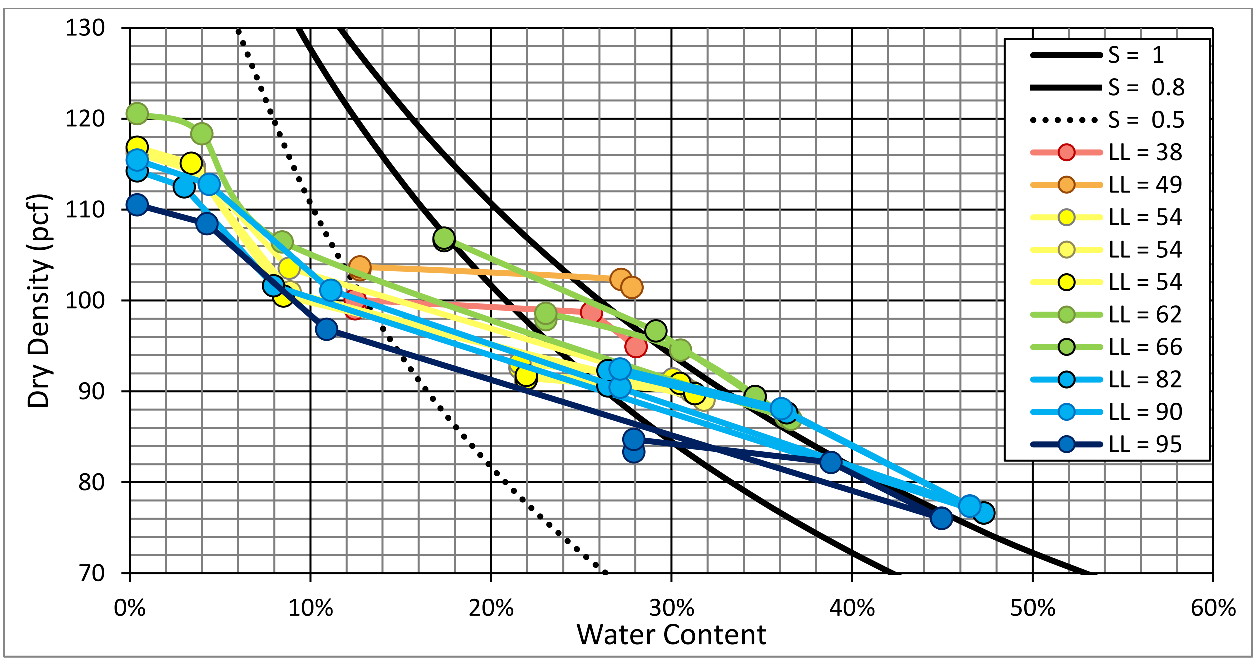


Figure 2.2.13: Swell-rebound-drying shrinkage paths for select specimens of FM 2 soils, filtered by swelling between 360 & 840 psf (3 & 7 ft.).

Changes in dry density correspond to the swelling strain according to the equation:

$$\epsilon = \frac{\Delta e}{1 + e_0} = \frac{e_f - e_0}{1 + e_0} = \frac{\gamma_{d_0}}{\gamma_{d_f}} - 1 \tag{2.3}$$

Where ε is the volumetric (swelling) strain,

e is the void ratio,

γ_{do} is the initial dry density,

and γ_{df} is the final dry density

The results of all of tests conducted using clay samples from the FM 2 site are presented in Figure 2.2.14 as vertical swelling strain plotted against a standardized depth below the ground surface assuming a total unit weight of 120 pcf. While a general positive trend is observed between swelling and liquid limit, the scatter in the data emphasizes the importance of selecting precise initial conditions, and also that the liquid limit may not be a precise predictor of the swelling of clays.

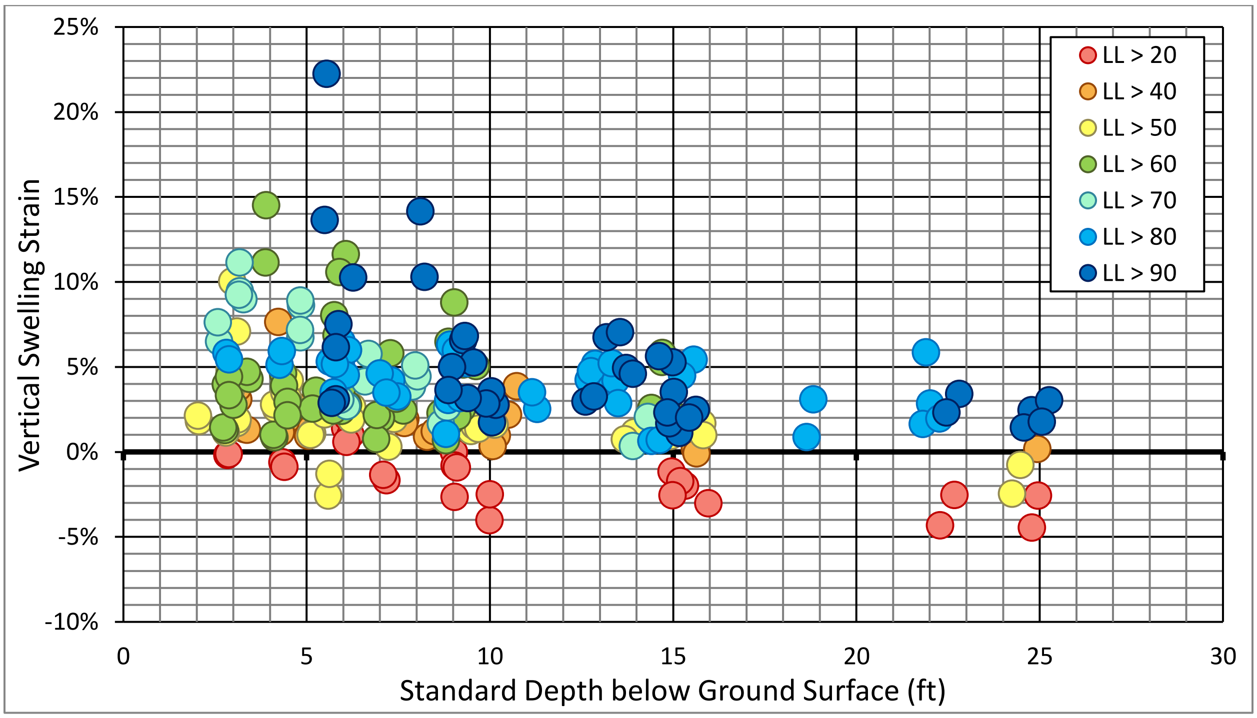


Figure 2.2.14: Swelling vs depth as obtained from centrifuge testing of moisture-adjusted specimens, grouped by liquid limit.

2.2.4. Effect of Binder Content

Soil binder is defined as the portion of a soil sample that passes a No. 40 Sieve, since it corresponds to the portion of the soil that tends to lend cohesion to the soil mass. The soil binder content is also largely responsible for the swelling properties of the soil. Soil samples from FM 972 were used to verify that swelling of a fine-grained soil decreases with increasing coarse fraction. The characteristics of the soils evaluated in this test series are summarized in Table 2.2.3. The four specimens from 8-10 feet depth have a higher plasticity, and a higher percentage of fines than the specimens from the top layer, which exhibit a somewhat lower plasticity, and have only 60% fines.

Table 2.2.3: Summary of soils for binder evaluation.

Soil	Specimens	Liquid Limit	Plasticity Index	Binder Percentage
FM 972 B2 0-2 ft	1,2,3,4	46	27	60%
FM 972 B2 8-10 ft	5,6	54	37	>80%

The soil specimens were first allowed to swell under the target load in the centrifuge and then rebound under a very small vertical load after stopping the centrifuge. Then the soil specimens were allowed to dry slowly and evenly, measuring the volume with a pair of calipers during the shrinkage stage. This process is shown conceptually in Figure 2.2.15.

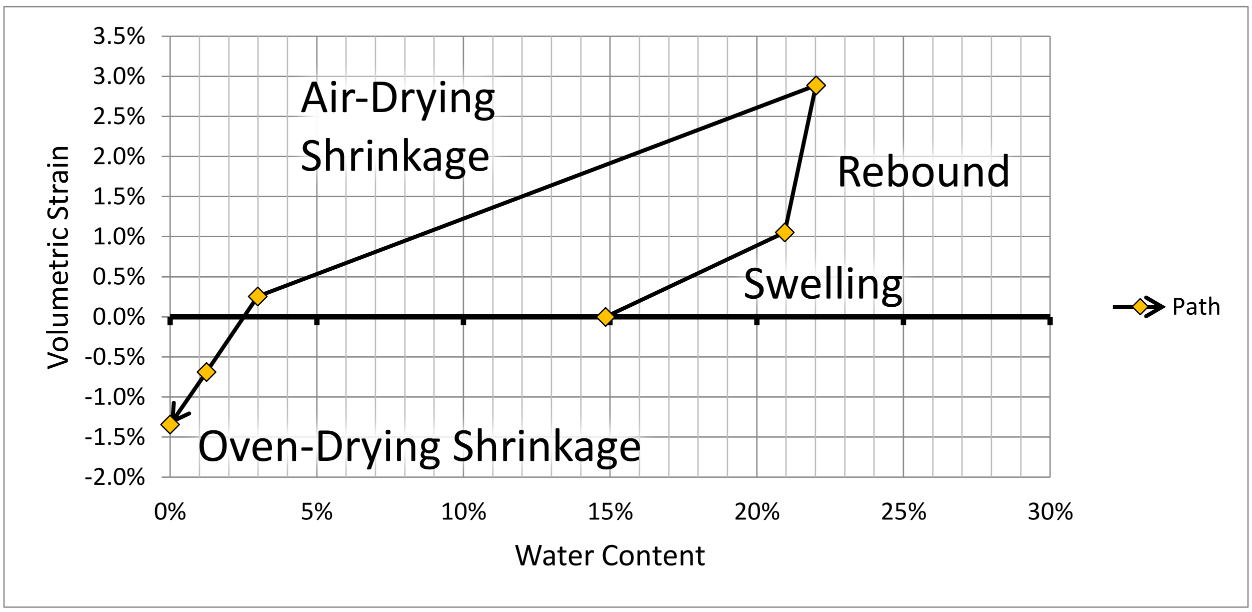


Figure 2.2.15: Conceptual strain path in single-cycle swell-shrink test.

Figure 2.2.16 shows the swelling-rebound-shrinkage paths of these specimens. The specimens with 60% binder swell somewhat less than the specimens with 80% binder, but the shrinkage upon drying is significantly less. This is expected, due to the restraining nature of the coarse particles in these former specimens. Figure 2.2.17 additionally shows these paths plotted as dry density vs water content for reference.

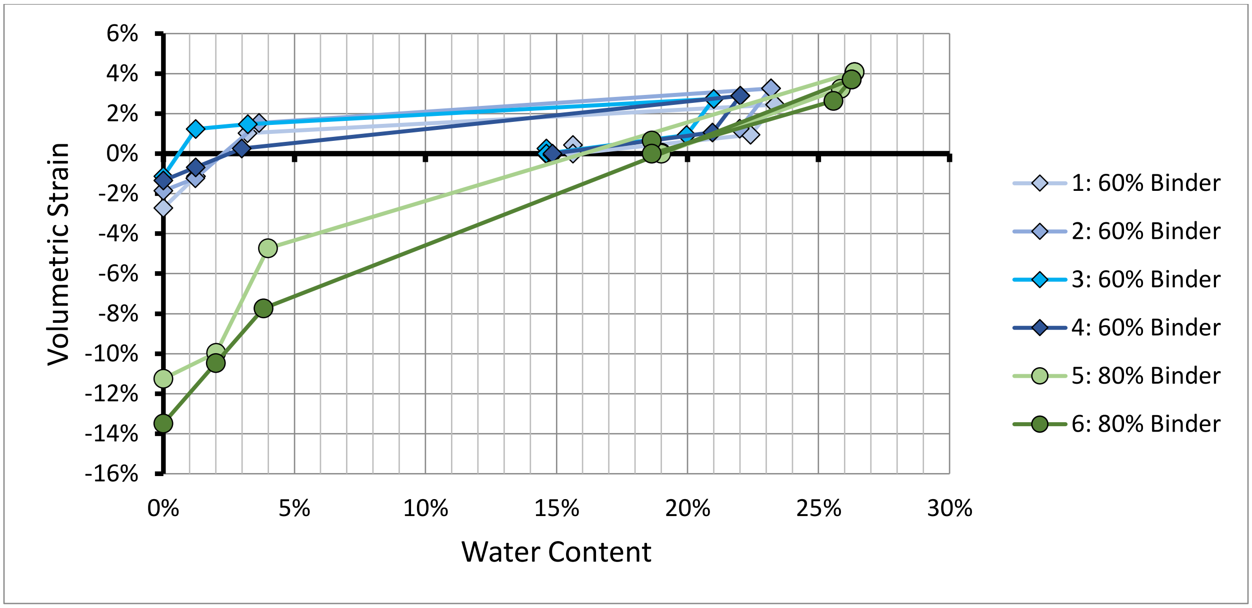


Figure 2.2.16: Swell-rebound-shrinkage strains for moderate plasticity specimens with and without significant coarse material.

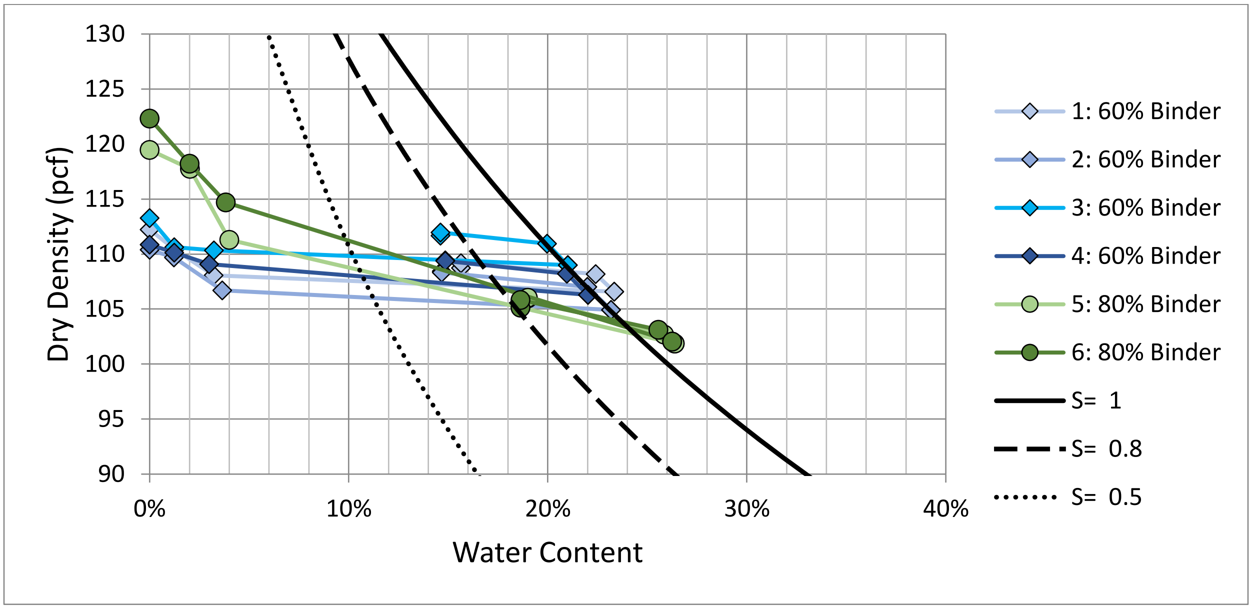


Figure 2.2.17: Swell-rebound-shrinkage path shown as dry density vs water content.

Furthermore, as shown in Figure 2.2.18, the soil samples with 80% binder swelled more than those with only 60% binder, regardless of the load applied on the specimens. This indicates that the presence of a significant coarse fraction will assist in preventing major volume changes during wet-dry cycles, and also that the centrifuge measurement can capture the exact effect of the presence of this coarse material upon the swelling in these soils.

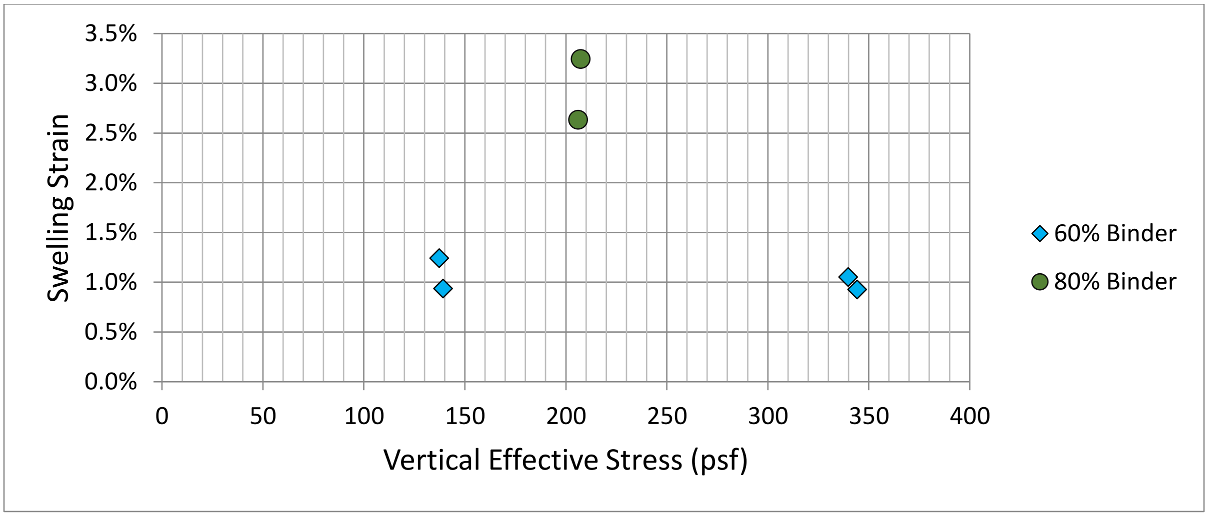


Figure 2.2.18: Swell-stress results for specimens from FM 972, showing decreased swelling due to presence of coarse fraction.

2.2.5. Conclusions from Baseline Testing Series on Untreated Soils

Evaluation of the swell test results from the baseline series on untreated soils leads to several conclusions. First, the initial conditions of the soil specimen are of the essence when predicting swelling upon wetting or shrinkage upon drying in expansive clays. Consequently, to establish a procedure by which to compare performance among different sites, it is recommended to always use the dry conditions when testing, but to make note of the actual conditions in the field, as some sites may not be prone to the full moisture fluctuations anticipated by this PVR model.

Second, the results from controlled testing on homogenized clays (Taylor Clay and Eagle Ford Clay) indicate that corrections to the measured swelling may be appropriate for small variations in moisture and density if the soil behavior is already well-known. Otherwise, it is advisable to control the initial conditions to within ± 1% in moisture and within ± 3.2 pcf of the target values.

Third, the effect of plasticity is evident in the swelling data, especially in light of the fact that clays of higher plasticity are usually prescribed to start the swelling test from higher initial moisture contents, and yet they can shrink significantly upon further drying. Had the specimens used in this testing program been prepared to drier conditions, they would have exhibited significantly greater swelling, based on the shrinkage results shown here.

Finally, the inclusion of a significant coarse fraction in the specimens does limit both the swelling and the subsequent drying shrinkage while specimens with no coarse fraction exhibit greater swelling and shrinkage. While this effect is addressed approximately within existing correlations, it may be a good practice to run tests on site-specific material when high-quality predictions are needed.

2.3. Results from Texas Swell Tests on Lime-treated Soils

2.3.1. Overview of Lime Treatment of Clays

One of the most widely-used methods to mitigate structural issues due to expansive clay soils is to stabilize the soil by adding lime. Two types of lime have often been used in soil stabilization—quicklime (CaO) and hydrated lime $Ca(OH)_2$. In the presence of additional water, each of these chemicals contributes to a similar set of reactions with the constituent silica and alumina of soil minerals. These reactions can be grouped into short and long-term interactions. Current design methods in use by TxDOT qualitatively account for both.

Figure 2.3.1: Increased friability in lime-treated clay (Carmeuse, 2018).

Two types of physical-chemical interactions govern the short-term stabilization of lime-treated clays. The first interaction involves cation exchange due to the introduction of larger cations. When the concentration of Ca^{2+} ions in the vicinity of the clay particles is increased by the addition of hydrated lime, the 'thickness' of the clay particles' diffuse double layer decreases, reducing the repelling force between individual particles and allowing them to pack more tightly.

This mechanism primarily is responsible for the reduction in swelling-upon-wetting in lime-treated clays. This is because a saturated solution of calcium hydroxide with excess reagents will tend to maintain a stable concentration as additional moisture is added. Thus, as the clay sample absorbs moisture, the concentration of dissolved Ca^{2+} should be maintained at relatively high levels, for adequate lime-treatment dosages.

Additionally, this increased concentration of Calcium ions in the pore-water solution will tend to displace other cations (such as Sodium, Na^+) from the negatively charged exchange sites on the surfaces of clay particles. The replacement of sodium by calcium tends to increase chemical stability, as Calcium has a larger electronegativity, and is thus harder to displace from the negatively-charged clay surfaces.

Another important effect of the increased charge per cation is allowing clay sheets to pack tightly enough, in some cases, such that the positively charged edges of the sheets may bond with the negatively-charged faces in a process known as flocculation. In this process, clay particles and hydrated Ca^{2+} ions form flocs, which effectively behave as larger soil particles. Figure 2.3.1 shows the visible impact this flocculation will have upon the properties of the clay: on the left-hand side is the original clay soil, while on the right-hand side is the lime-treated soil. The lime-treated soil resists full hydration due to the reduction of the clay particle's diffuse double layer and the formation of flocs, resulting in a less plastic and more friable soil under the same initial conditions.

The second short-term interaction, which may tend to counteract the effects of the dissolved cation concentration is a process called carbonation. Carbonation occurs as the hydrated lime reacts with gaseous carbon dioxide to form Calcium Carbonate. This reaction has been reported to produce cementation in porous materials (Moorehead, 1986) but the overall impact in a low-permeability soil with minimal access to gaseous CO_2 may be minimal.

A long-term effect of the addition of hydrated lime is to increase the OH^- ions in solution, leading to an increase in pH of the system (which is often used as a design criterion in lime-stabilized soils). As pH exceeds about 12.4, the clay molecules begin to break down and the silica and alumina become soluble and subsequently react with calcium ions to produce cementitious materials: calcium silicate hydrate and calcium aluminate hydrates. These reactions are referred to as pozzolanic reactions and often require long periods of time to occur.

In addition to the lime dosage, two important determinations of the stabilization process are those of the mellowing time and the curing time. Mellowing is defined as the period between soil-lime mixing and compaction, whereas curing is defined as the period after final compaction before further site construction.

A mellowing time from 1-4 days is included in Tex-112-E to allow sufficient time for the soil, lime, and water to mix and produce a more homogenous and friable material that is then much easier to mix and compact for construction.

However, the literature is less clear on whether the mellowing time has an adverse effect on the compressive strength of the soil-lime mixture. In general, it is seen that mellowing times decrease the compressive strength of soils but may or may not increase the potential swelling of these soils (Mitchell and Hooper, 1961; Belchior, 2016).

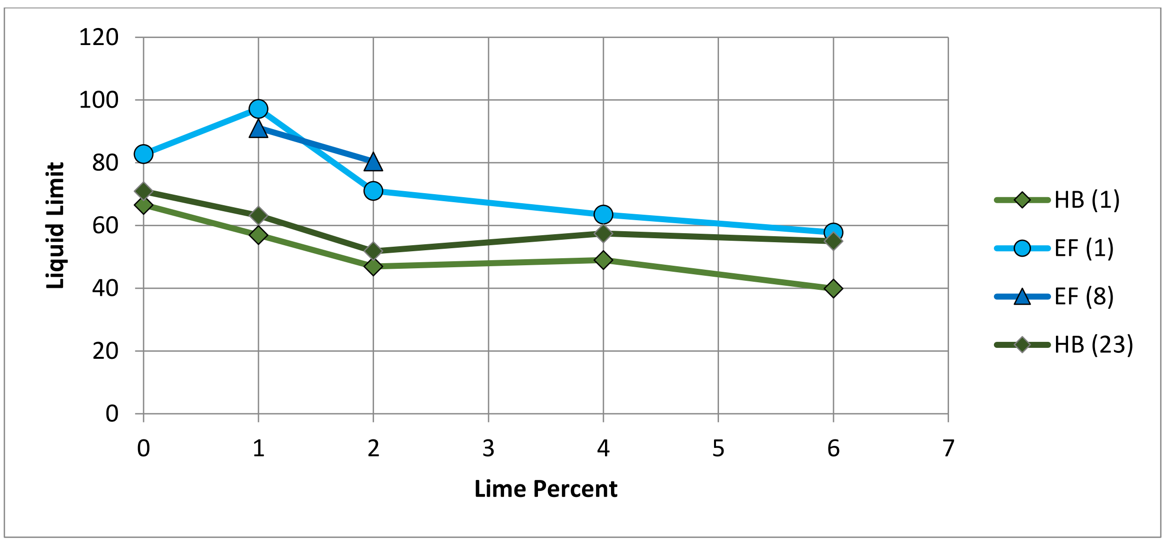
However, the procedures documented by TxDOT tend to include a mellowing time in their test procedure in addition to construction specifications, so any sizeable decreases in compressive strength are likely taken into account. Additionally, an increased mellowing period may be beneficial to treated soils with a high sulfate content to allow any detrimental reactions (such as the formation of ettringite) to proceed before compaction and strength testing.

Curing, on the other hand, is generally performed to allow some of the pozzolanic reactions to occur and allow the soil to gain additional strength before final construction. Generally, the soil shear strength increases throughout the curing period, while the swelling potential is expected to decrease.

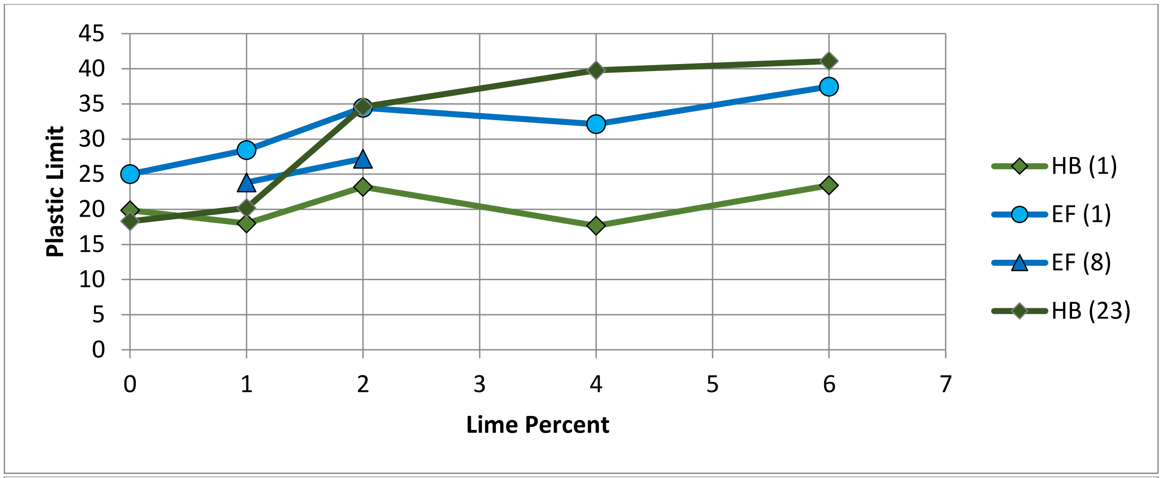
In order to directly quantify the impact of lime treatment upon the swelling behavior in expansive clays, a series of swell tests using centrifuge technology were performed on the Eagle Ford Clay: a high-plasticity clay from I-35 & Hester's Crossing, in Round Rock, Texas. Results of the tests performed on Eagle Ford Clay are presented in Chapter 2 and Chapter 3 of this report. General characterization tests were also performed on Houston Black clay with and without mellowing time, which will also be presented in this chapter.

2.3.2. Effects of Lime Treatment upon the Index Properties of Expansive Clays

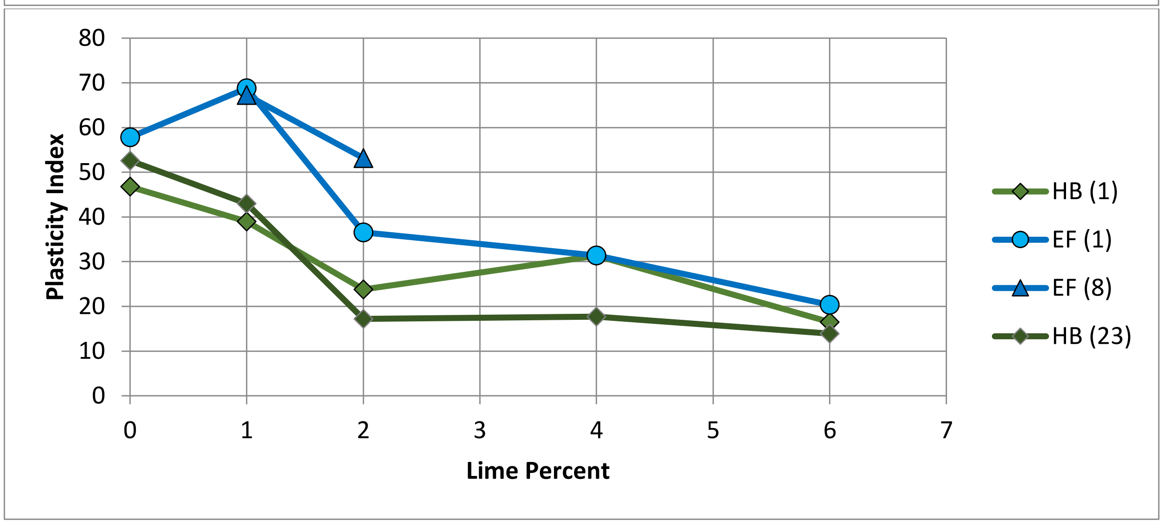
Atterberg limit tests were performed on two clay materials before and after treatment with varying dosages of hydrated lime, to observe the qualitative effects of the treatment. Samples of the treated soil were tested again after a number of days to observe the effects of mellowing time upon the result. Increasing dosages of hydrated lime tended to reduce the liquid limit and increase the plastic limit as shown in Figure 2.3.2. This alteration converts the index behavior of the soil from clay-like to silt-like, and from high plasticity (LL>50) to low plasticity (LL<50) as shown in the plasticity chart for these soils in Figure 2.3.3. At very small dosages of lime, the liquid limit is observed to increase slightly in some cases, possibly due to better dispersion of the clay particles during the preparation of the test. Caution should be used in interpreting this apparent increase, however, as the source shale for these tests is itself a layered material with visibly different seams, and repeated Atterberg Limit testing on identical soils by even the same operator may be subject to scatter on the order of 5%.



(a)



(b)



(c)

Figure 2.3.2: Index parameters on natural and lime-treated clays: (a) liquid limits; (b) plastic limits; and (c) plasticity index.

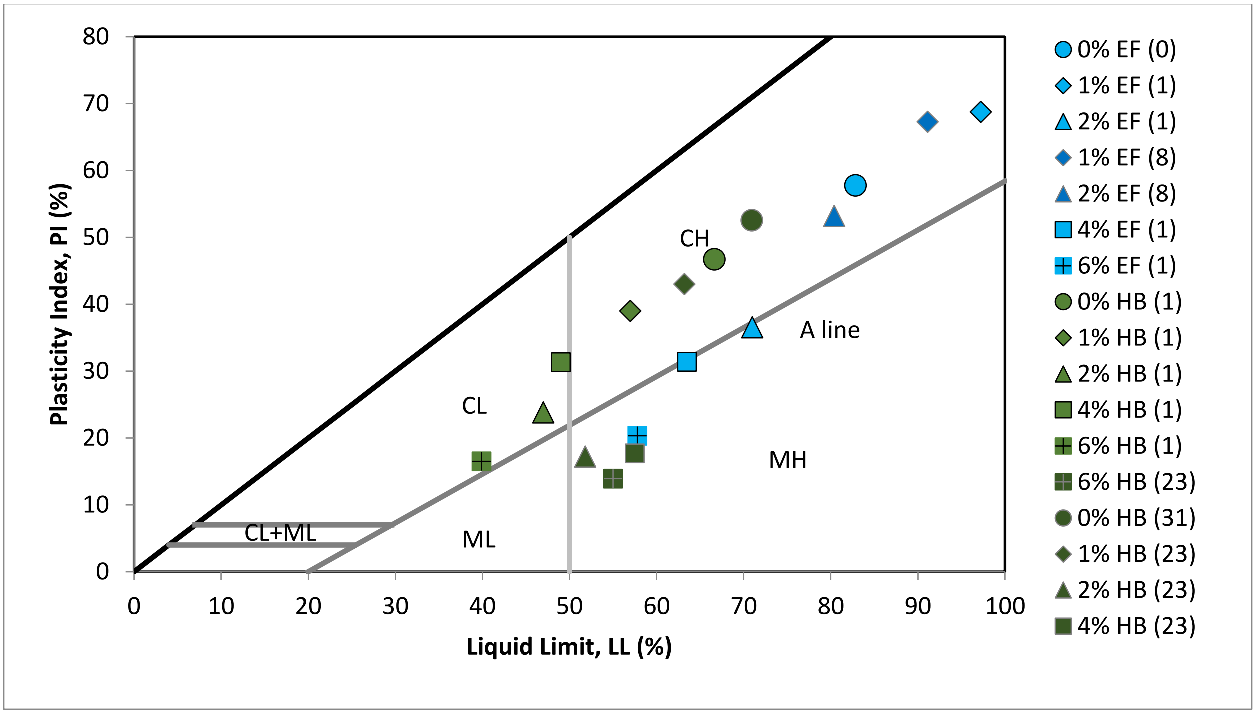
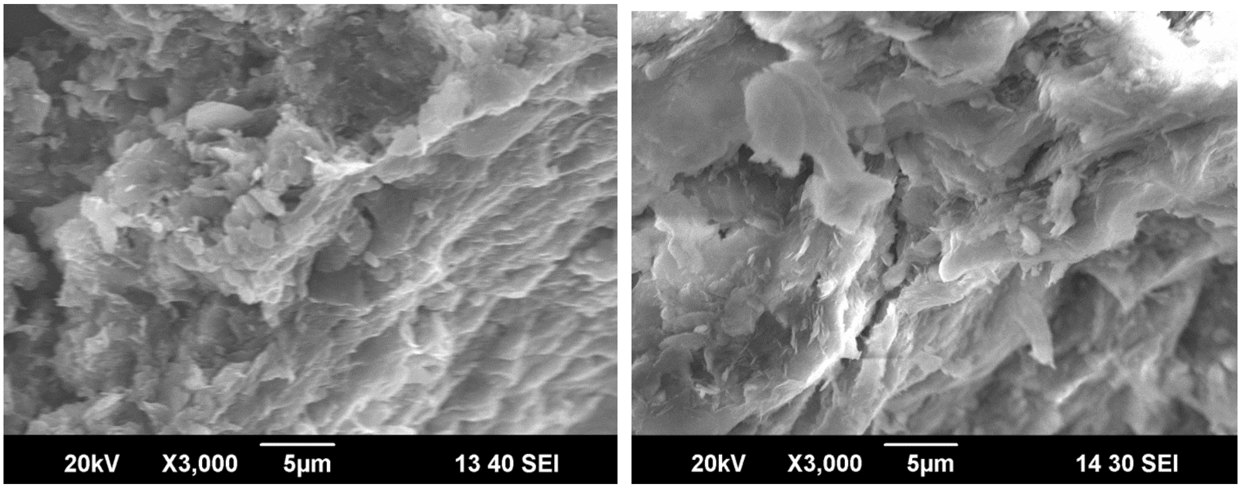


Figure 2.3.3: Plasticity chart showing effect of hydrated lime on Houston Black Clay A horizon soil (HB) and Eagle Ford Clay (EF) (labeled percentages are mass percentages of hydrated lime by dry mass of soil solids; values in parentheses are days of mellowing prior to testing).

2.3.3. Scanning Electron Microscopy on Treated Samples

Figure 2.3.4 shows images of Eagle Ford clay treated with 6% and 4% hydrated lime, respectively. These samples had been subjected to no curing time at the time of collecting the images. At this dosage of treatment, no significant structural difference is observable in the images, indicating that the primary effects of treatment at this stage are likely only manifested in the presence of water as diffuse double layer interactions.



(a) (b)

Figure 2.3.4: Scanning electron microscope images of Eagle Ford Clay: (a) treated with 6% hydrated lime; and (b) treated with 4% hydrated lime.

2.3.4. Eagle Ford Treated with 2% Hydrated Lime

Centrifuge swell tests were performed on lime-treated Eagle Ford Clay specimens treated with a dosage of 2% based on the mass of soil solids. Table 2.3.1 shows the range of compaction conditions for specimens tested, and Figure 2.3.5 shows the swell-stress results from each test. The data shows a decreasing trend in swelling with increasing effective stress. As a reference, the swelling at 100 psf is about 6% ± 2%. Two different curves are considered to represent the result trends: a Log-linear trend line, and a 3-parameter trend line. Table 2.3.2 shows the R^2 values for each model; the low values indicating that while both models capture the overall trend in the data, significant scatter exists in relation to the trend line.

Table 2.3.1: Range of compaction conditions for 2% lime-treated Eagle Ford Clay swell data.

	Moisture Content	Compaction Void Ratio	Dry Density [pcf]
Minimum Value	0.230	0.785	93.7
Maximum Value	0.249	0.833	96.7
Percent Error	8.1%	6.2%	3.2%

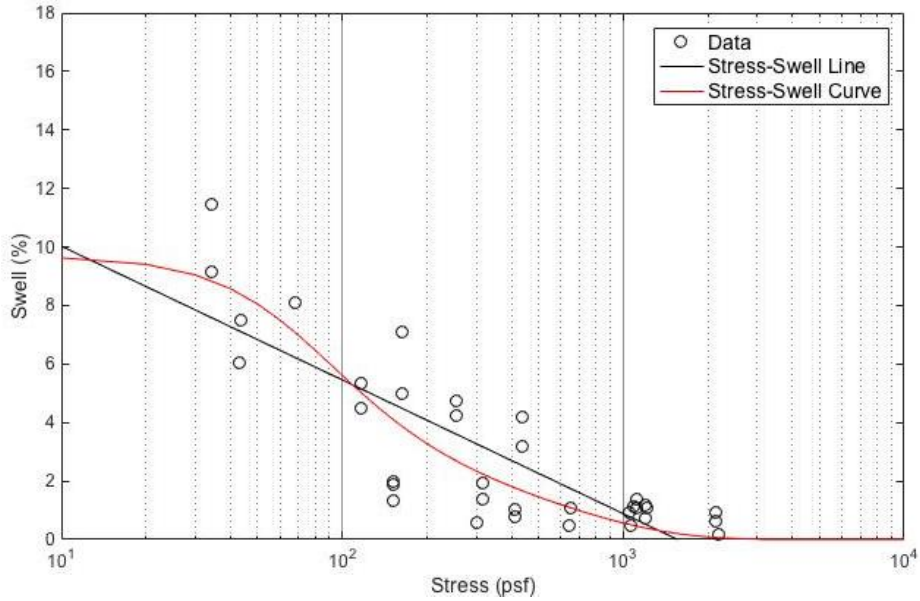


Figure 2.3.5: Swell results for Eagle Ford Clay treated with 2% hydrated lime (trend lines also shown as reference).

Table 2.3.2: R2 values for best-fit curves for 2% lime-treated Eagle Ford Clay.

R2 for 3-Parameter Curve	R2 for Semi-log Line
0.7583	0.6943

2.3.5. Eagle Ford Treated with 4% Hydrated Lime

A set of tests was also performed on Eagle Ford treated with 4% hydrated lime. The range of compaction conditions for these specimens is given in Table 2.3.3 and the swell-stress data is presented in Figure 2.3.6. This data exhibits more scatter than the untreated Eagle Ford samples, but the swelling magnitude is also significantly lower. At this treatment dosage and initial moisture content, the average swelling at 100 psf is about 2% ± 2%.

R^2 values for each of the model curves are given in Table 2.3.4. In this case, the 3-parameter curve approximates a semi-log-linear trend line. Again, the small R^2 value indicates that there is significant scatter around the trend line.

Table 2.3.3: Range of compaction conditions for 4% lime-treated Eagle Ford Clay swell data.

	Moisture Content	Compaction Void Ratio	Dry Density [pcf]
Minimum Value	23.0%	0.792	89.6
Maximum Value	25.0%	0.911	95.7
Percent Error	8.5%	15.1%	6.7%

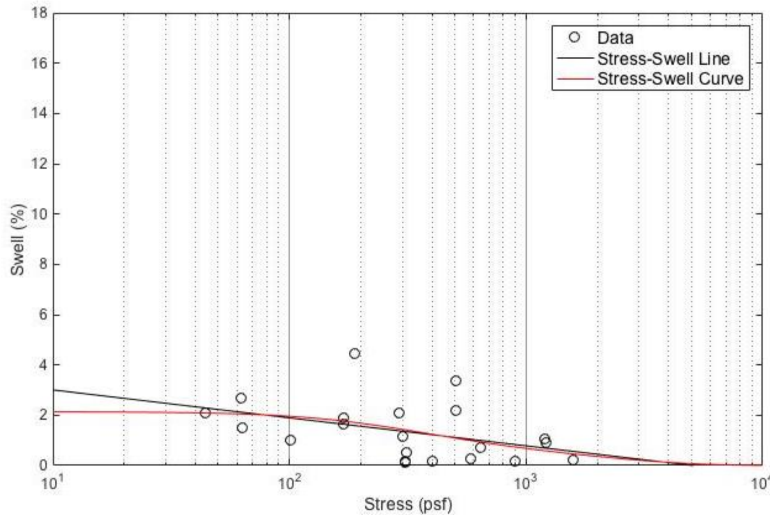


Figure 2.3.6: Best-fit curve and best-fit line for Eagle Ford Clay treated with 4% hydrated lime.

Table 2.3.4: R^2 values for best-fit curves for 4% lime-treated Eagle Ford Clay.

R2 for 3-Parameter Curve	R2 for Semi-log Line
0.1789	0.1713

2.3.6. Lime-Treated Eagle Ford Swell-stress Curves

Additional centrifuge swelling tests were performed on Eagle Ford Clay treated with dosages of 1 and 6 % hydrated lime. The combined swell-stress data from treated Eagle Ford clay is shown in Figure 2.3.7 along with the swell-stress data from the untreated soil. This data indicates that the swell-stress curves can be adequately represented with a linear relationship with the logarithm of stress, and that 95% of the data fit within approximately ± 3% strain of the mean trend for any given treatment dosage. While the data for 6% lime technically allows a slightly positive relationship between swelling strain and effective stress, a negative or zero trend would be a more reasonable interpretation of the data, and would fit the data nearly as well. Additionally, the relatively steep slope of the data at 1% lime may be due to the relative lack of data at this lime percentage. In general, the primary influence of lime treatment (all other variables being the same) is in the reduction of the slope of the swell-stress line. As discussed previously, this can be attributed to diffuse double layer suppression in the presence of the cations contributed by the lime, causing the soil to become less sensitive to changes in moisture.

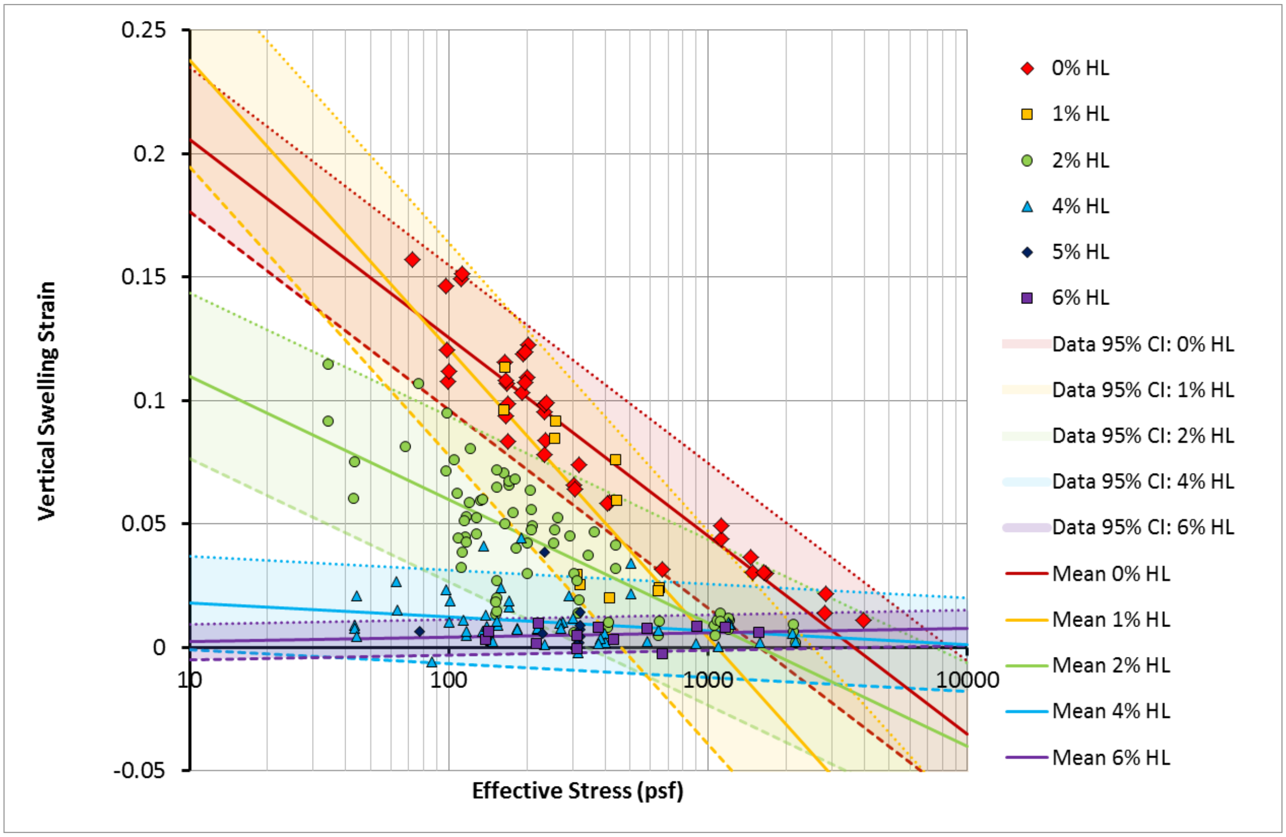


Figure 2.3.7: Swell-stress curves from tests using Eagle Ford Clay specimens grouped by lime dosage.

2.3.7. Effect of Mellowing Time

Mellowing refers to the common construction process that involves allowing a soil-lime mixture to rest for a period of time *before* compaction. Previous research has found that allowing soil to mellow for more than approximately 12 hours leads to an increase in swell (Belchior, 2016). This is potentially because pozzolanic reactions begin to occur during the mellowing period and some of the soil beings to ‘cement” together, and then these soil bonds are effectively crushed during compaction, negating some of the effects of the lime. The effect of mellowing time was evaluated experimentally in this project using Eagle Ford clay specimens treated with 2% and 4% by mass of hydrated lime over a range of stresses. Samples were mixed and allowed to rest in sealed plastic bags before being compacted and tested in the centrifuge.

Figure 2.3.8 shows the effect of 2% lime-treated soil after mellowing for 1 day, 28 days, and 43 days. The effect of mellowing on results, at least for a lime dosage of 2%, is not significant—the mellowed samples swelled within the margin of error of the samples that were immediately mixed and tested. Figure 2.3.9 shows the effect of 4% lime-treated soil after mellowing for 1 day, 6 days, and 43 days. Again, there is not a noticeable effect on the swelling magnitude due to mellowing time, particularly within the inherent scatter of lime-treated soil samples.

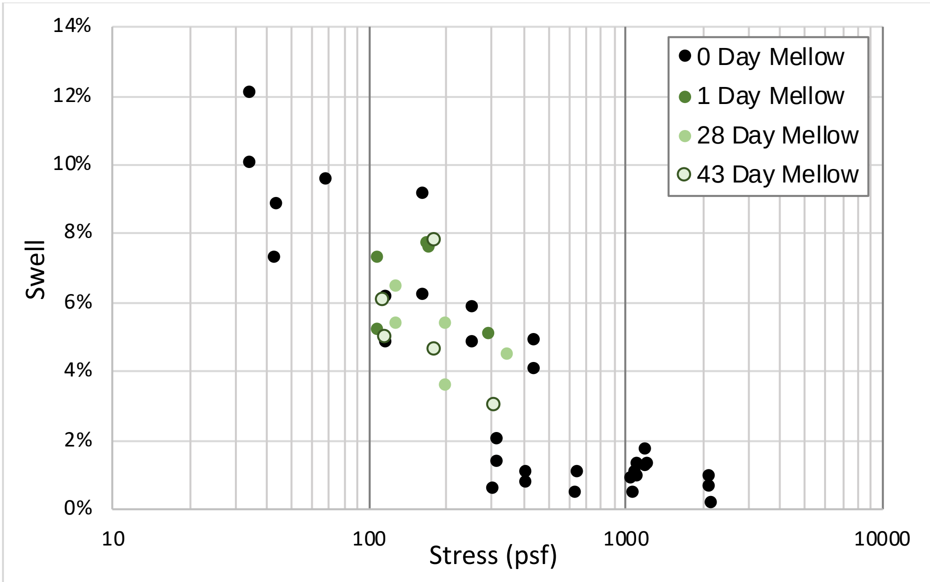


Figure 2.3.8: Variation in mellowing time for Eagle Ford Clay treated with 2% hydrated lime.

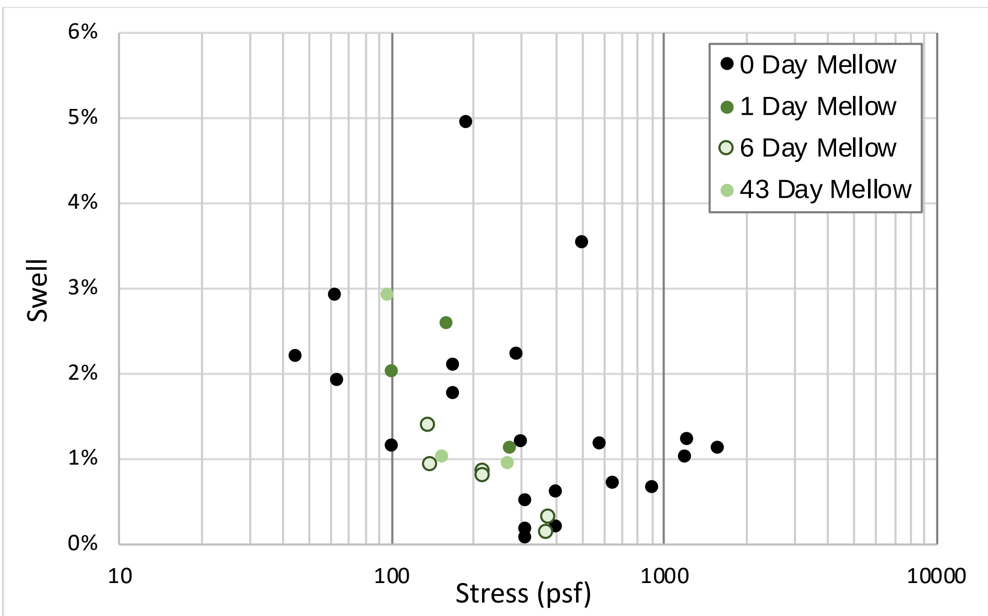


Figure 2.3.9: Variation in mellowing time for Eagle Ford Clay treated with 4% hydrated lime.

2.3.8. Effect of Curing Time

Curing refers to the time that lime-treated soils are allowed to rest *after* having been compacted. It is generally recommended that lime-treated soils be allowed to cure for at least 4 weeks to allow pozzolanic reactions to fully develop, as this allows for a significant decrease in swell potential and an increase in compressive strength. However, these reactions may not fully occur if enough lime is not used. To assess the effect of curing time, a series of swell-stress curves were generated using results from tests conducted with Eagle Ford specimens treated with 2% and 4% hydrated lime. Samples were compacted in a 2.5-inch cutting ring and allowed to cure in a fog room for the prescribed amount of time. At the time of testing, specimens were trimmed to fit in the 2-inch cutting rings and tested in the centrifuge.

Figure 2.3.10 shows the swell-stress data for 2% lime-treated Eagle Ford without curing and after curing for 14, 21, and 56 days. No significant decrease in swell is observable in the cured specimens of 2% lime-treated Eagle Ford. This is likely because a dosage of 2% is insufficient to properly activate the pozzolanic reactions in the highly plastic Eagle Ford soil.

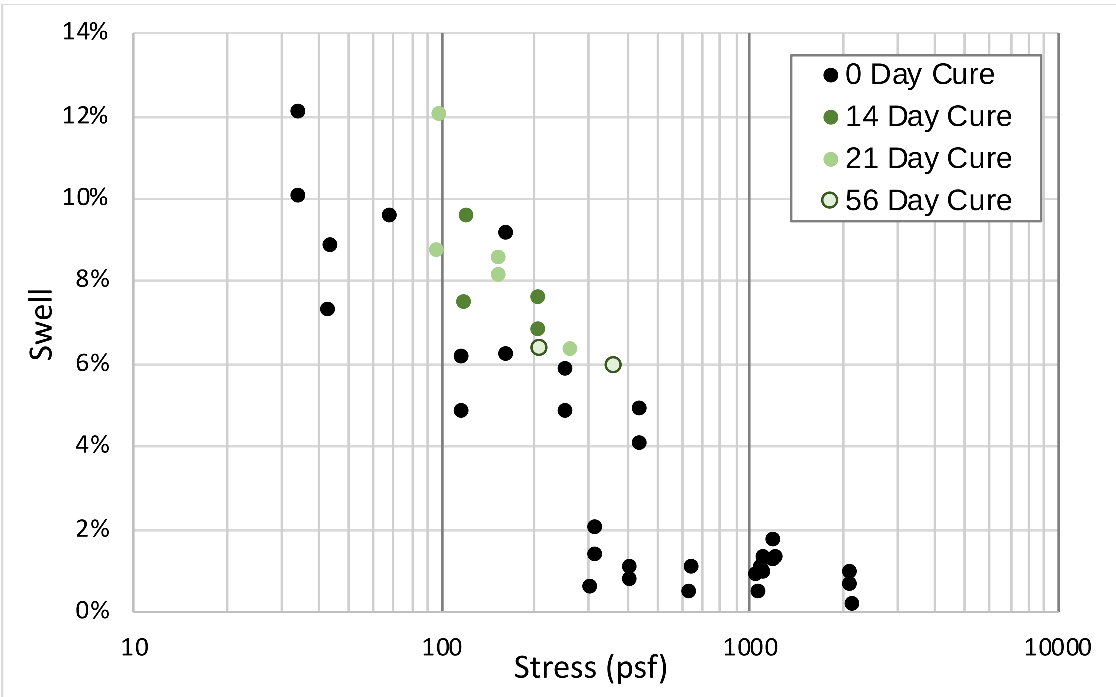


Figure 2.3.10: Effect of curing time on the swell of Eagle Ford Clay treated with 2% hydrated lime.

Figure 2.3.11 shows the swell-stress curves for 4% lime-treated Eagle Ford without curing and after curing for 42 and 56 days. In this case, the increased curing time significantly reduces the swelling magnitude in the specimens. However, it is unlikely that standard testing schedules will allow for samples to cure for 4-6 weeks before testing, so it is likely not an efficient use of time to prepare many cured samples for testing lime-treated soil specimens.

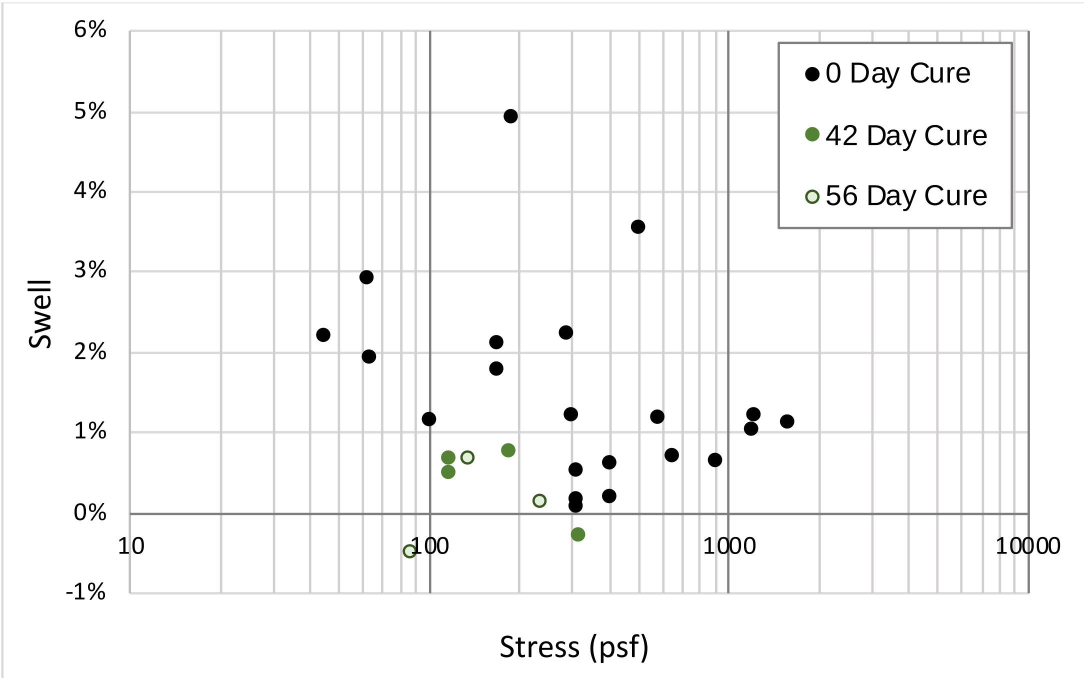


Figure 2.3.11: Variation in curing time for Eagle Ford Clay treated with 4% hydrated lime.

2.3.9. Quantification of the Swell Pressure

Because both the treated and untreated swell-stress curves can be characterized using log-linear trends, it is convenient to attempt to describe the data in terms of a single zero-crossing along the stress-axis (that is, the 'swell pressure'), and a single logarithmic slope. However, previous research focused on reductions in swell pressure with the addition of hydrated lime has indicated that the swell pressure may indeed decrease with additions of hydrated lime. To assess the validity of this assumption, additional tests were performed to compare possible interpretations of the soil swell pressure.

A free swell test was performed on Eagle Ford treated with 4% hydrated lime. The test was prepared in accordance with ASTM D4546 Method C, also called the *loading after wetting* test method. In this method, the specimen was inundated and allowed to swell under a stress of 250 psf. After swelling, the load on the specimen is increased, similar to a standard consolidation test, and the load-induced strains are measured. The swell pressure is determined as the pressure required to revert the specimen to its initial thickness. While this is not necessarily expected to yield the same swell pressure as a wetting after loading test, the value should provide an upper bound to the possible swell pressure for this soil.

Figure 2.3.12 compares the swelling magnitude of this specimen to other specimens of Eagle Ford treated with 4% lime and prepared to similar initial conditions. As seen in the figure, the free swell data matches well with centrifuge swell data for this case.

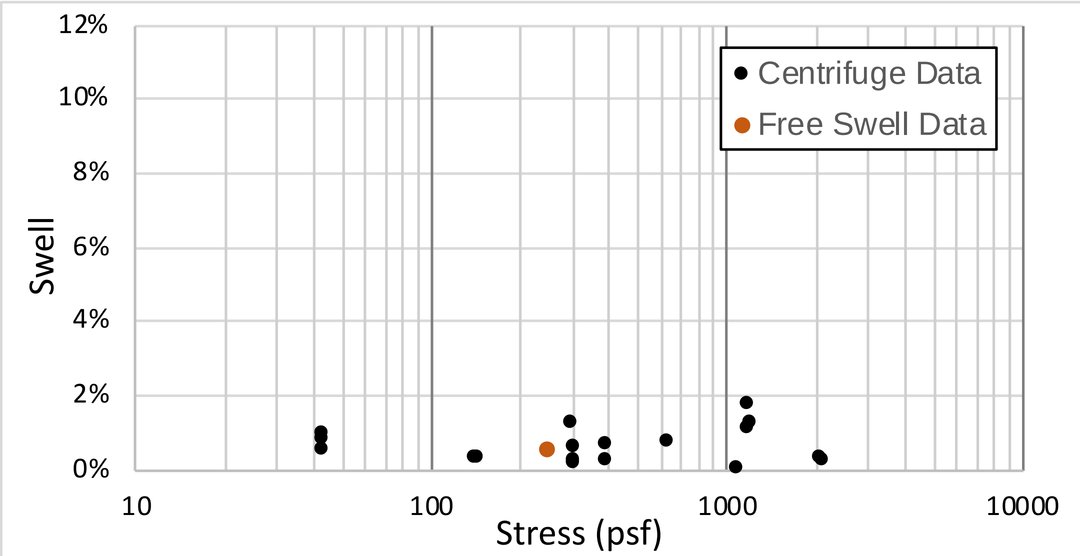


Figure 2.3.12: Centrifuge and free swell test results for Eagle Ford Clay treated with 4% hydrated lime.

Figure 2.3.13 shows the results of the swell-consolidation test. The results show an approximately linearly decreasing strain with increasing applied stresses. A "swell pressure" of approximately 2100 psf can be defined from the data.

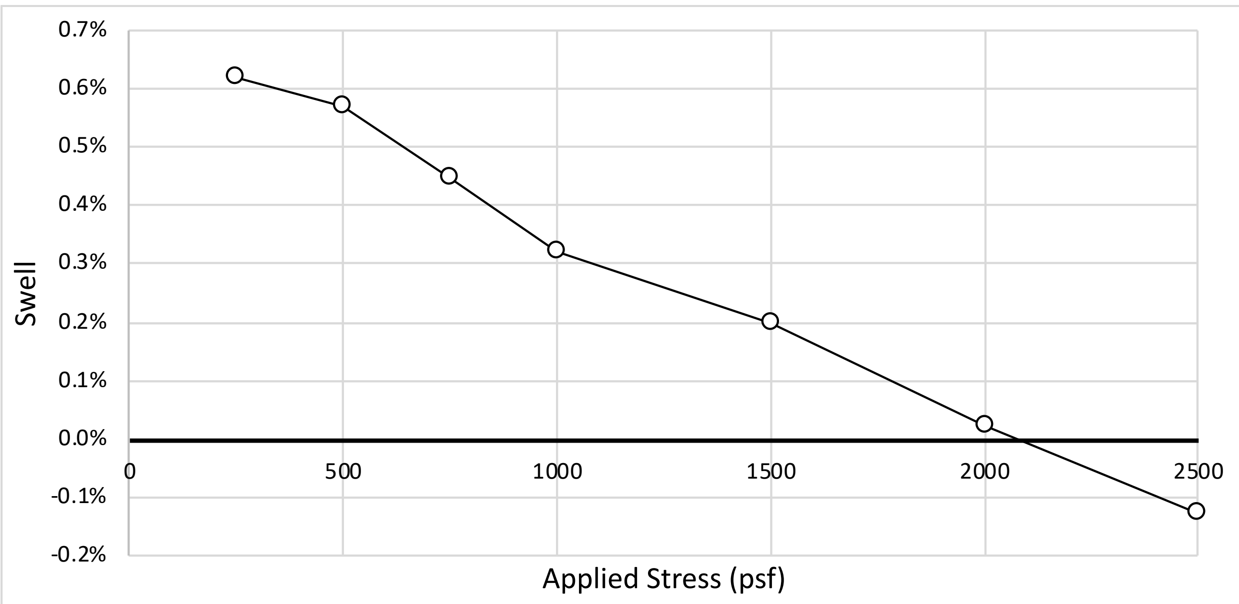


Figure 2.3.13: Swell-consolidation test results for Eagle Ford Clay treated with 4% hydrated lime.

The swell pressure value obtained from free swell testing was compared to swell pressures determined based on the trend defined from other methods:

- the 3-parameter curve fit to the 4% lime centrifuge data,
- the logarithmic trend line fit to the 4% lime centrifuge data, and

- the extrapolated swell pressure of the logarithmic trend line fit to the untreated Eagle Ford data.

The results are tabulated in Table 2.3.5 and shown graphically in Figure 2.3.14. It can be seen that the 'swell pressure' as such, is an ill-identified zero-crossing which may span a range of several thousand psf, depending on the graphical method used to derive the value. This determination may be particularly difficult if the dataset involves significant scatter. Consequently, the adopted assumption adopted is that the swell pressure can be defined using the results from untreated clay specimens as the scatter is expected to be smaller, although it is recognized that its use is primarily as a fitting parameter to optimize the number of tests required when evaluating chemical treatment of clays.

Table 2.3.5: Comparison of swell pressure values for 4% lime-treated Eagle Ford Clay from graphical methods.

Free Swell Pressure [psf]	3-Parameter Curve Swell Pressure [psf]	Semilog Linear Swell Pressure [psf]	Untreated Semilog Linear Swell Pressure [psf]
2100	3000	5500	3500

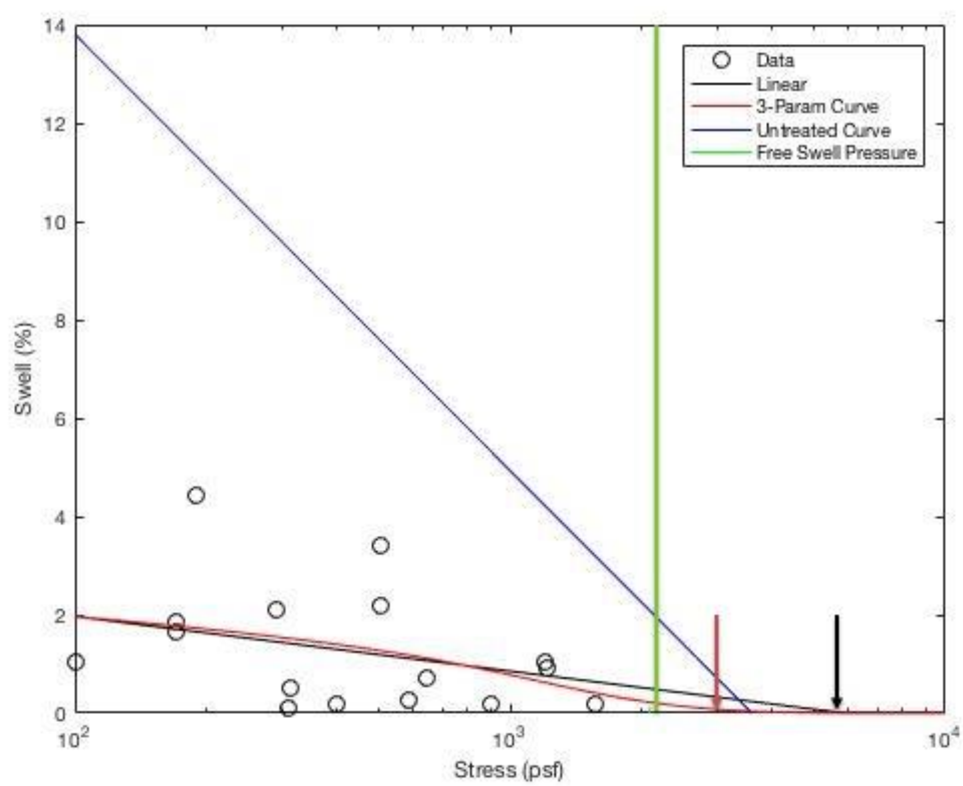


Figure 2.3.14: Comparison of swell pressure values for 4% lime-treated Eagle Ford Clay from graphical methods.

2.3.10. Evaluation of Aggregated Swell Test Data

A summary of the major findings from the lime-treated swelling program is presented in this section. Swelling results were selected across the full range of lime-treatment dosages used in the testing program in order to demonstrate the selection of an optimal treatment dosage for this soil.

Figure 2.3.15 shows the swelling strain plotted against the lime percentage in each specimen for tests conducted within a comparatively narrow window of stresses ranging from 309 to 337 psf. Table 2.3.6 shows the range of compaction conditions for these specimens.

The addition of hydrated lime significantly decreases the swelling potential for this soil up to a dosage of about 3%, beyond which the soil does not swell significantly.

Table 2.3.6: Range of compaction conditions for lime-treated Eagle Ford Clay swell data.

	Moisture Content	Compaction Void Ratio	Dry Density [pcf]
Minimum Value	0.232	0.793	93.6
Maximum Value	0.247	0.844	95.5
Percent Error	6.5%	6.5%	2.1%

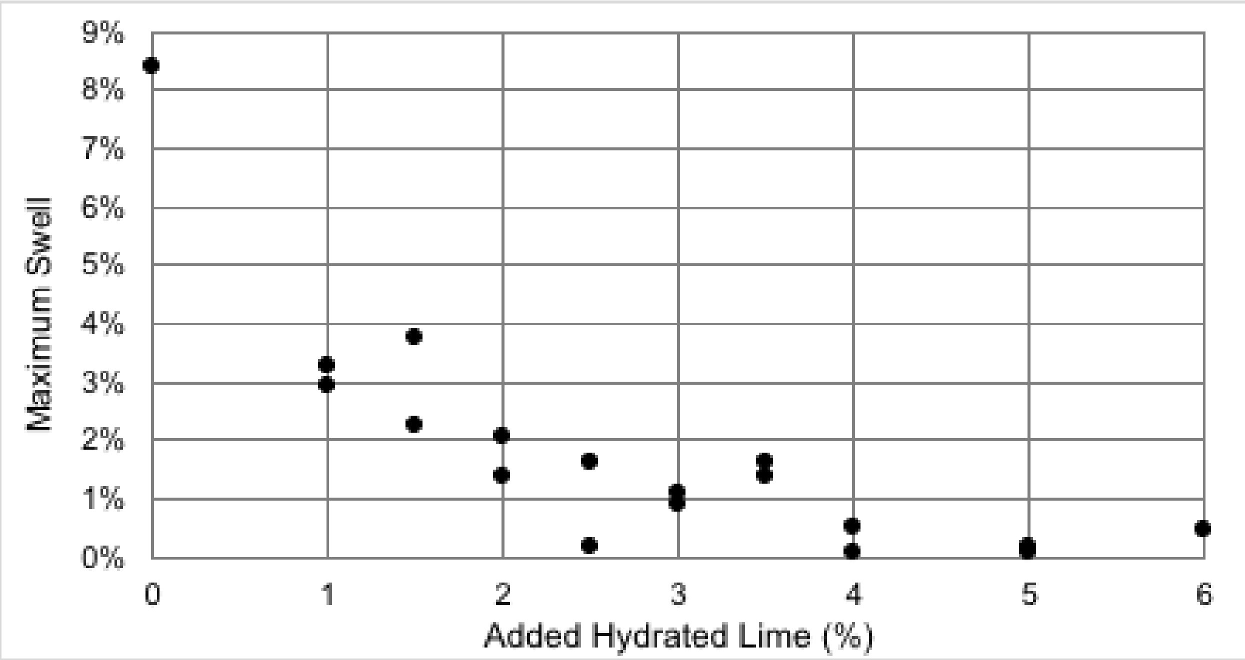


Figure 2.3.15: Variation of swelling magnitude with lime dosage for Eagle Ford Clay at 300 psf.

2.3.11. Conclusions from Lime Treatment Testing Series

The following general conclusions may be drawn from the evaluation of lime-treated highly plastic clays, as follows:

- The addition of hydrated lime in dosages of 6% or less produces a noticeable change to the index behavior of highly plastic clays evaluated in this study. In particular, a dosage of 3% or more essentially eliminated swelling.

- While curing may have a beneficial impact upon the swelling of clay samples after long periods of curing time, the benefits gained during the curing process largely fall within the range of scatter of un-cured specimens. Thus, for practical planning purposes, curing may not be necessary prior to swell testing.

- Mellowing did not significantly affect the amount of swelling. Because allowing the soil to mellow for approximately 24 hours does produce a more workable mixture, it is recommended that the moisture conditioned soil-lime mixture be prepared and allowed to rest for 12-24 hours before testing.

- Evaluation of the 'swell pressure' using extrapolation of the trend of swell-stress curves is feasible but has led to scatter. Consequently, an approximate value can be obtained from the swell-stress curve of untreated soils but only for use as a fitting parameter to define log-linear fit lines of lime-treated soils.

Chapter 3. Sensitivity of Testing Variables on Centrifuge Test Results

3.1. Testing Protocols

Testing protocols for the centrifuge swelling test, identified herein as the Texas Swell Test have been developed as part of previous projects, as reported by Zornberg et al. (2013, 2017). Refinements to these previously developed protocols were considered throughout the course of this project, particularly with the objective with the objective of making preparation of test specimens comparatively more uniform and efficient as well as of minimizing scatter of test results.

In addition, the original testing protocols have been extended to also include sample preparation and testing of lime-treated specimens, so that direct comparisons of swelling could be made between natural and treated soils.

The final recommendations for the test procedures have been compiled in the form of a draft procedure for the Texas Department of Transportation (TxDOT). These draft procedures may be found Appendix A.

Several important considerations have been considered in the development of the refined protocols for the Texas Swell Test. These include:

- The impact of initial dry density and initial moisture content on the swelling results. Based on results of the field sampling program and subsequent swell-shrink tests on expansive clays, the density and moisture content were found to be strongly interdependent within the normal range of swelling. Additionally, the in-situ density in highly expansive clays under sufficient confinement is thought to correspond with a high degree of saturation (S_r > 85%) when the in-situ water content is above the shrinkage limit. Consequently, the initial conditions for testing used in this method are prescribed in terms of moisture content and density to specifically achieve such a degree of saturation.

- Default values for initial soil density and initial moisture content were established in the refined testing protocols. While it is recognized that the designer may end up specifying values other than the default ones, establishing default initial conditions is expected to lead to consistency in test results and clearer implications of PVR values adopted as design criteria. Simple correlations were identified to predict the initial moisture content and corresponding soil density to be adopted for centrifuge testing.

- Swell-stress data generated for multiple soil types tested using normal surcharge values ranging from 100 to 1000 psf (representing the top 20 feet in typical soil deposits) indicate that a log-linear curve fitting is adequate to reliably represent swell-stress curves.

Additionally, the use of log-linear functions was found to allow PVR calculations to be conducted in a comparatively simpler manner.

- When assessing the impact of lime treatment in low-overburden layers (near surface layers) the reduced slope of the swell-stress curve was found to be a more reliable indicator of the treatment than the predicted swell pressure (the zero-crossing of the curve) from the same data. However, the testing protocols recommend a minimal testing program based on the assumption that the swell pressure is the same for natural and treated soils and that it can be consequently from the natural soil swelling curve. This assumption greatly facilitates the prediction of the swell-stress curve on lime-treated clays as it minimizes the number of experimental data points that should be generated.

3.2. Evaluation of the Sources of Variability in Centrifuge Test Results

As will be further discussed in Section 3.2.6, the potential swelling of a soil depends significantly on its mineral composition, and its initial conditions (particularly initial dry density and moisture content). However, tests on apparently similar specimens have shown scatter in test results that often show differences in swell strain values as high as 2 to 3%.

While some of this variability is expected to be due to the heterogeneity of the soil sample (e.g., actual minerals and pore structure of each soil specimen), it was also anticipated that some of the results' variability may be due to inconsistencies or differences that occur during sample preparation and testing procedures.

Consequently, as part of the effort aimed at refining the testing protocols for the Texas Swell Test, a parametric experimental program was conducted specifically to assess the sensitivity of the swell test results to several of the identified parameters and variables established in testing protocols.

Variables considered in this evaluation included:

- The use of lubricant (vacuum grease) on the walls of the testing ring with the objective of reducing sidewall friction
- The method by which soil samples were dried (air vs oven-drying) during pre-processing of centrifuge swelling test specimens
- the height of water ponded above the specimen at the beginning of centrifuge testing
- the target thickness to be specified for soil specimens
- the precision with which the target specimen thickness is achieved during specimen preparation

- and the alternative ways by which a target vertical effective stress can be achieved during centrifuge testing (namely the selected surcharge mass or centrifugal acceleration to apply to the specimen)

Specimens used in this evaluation were prepared and tested in accordance with the testing protocols laid out in Appendix A, except insofar as the testing parameters were varied specifically to examine their impacts upon the swelling measurement. The ranges of the initial conditions for each test series are reported along with the results.

3.2.1. Effect of Using Lubricant (Vacuum Grease)

Vacuum grease had been routinely used on the inside of the cutting rings of the test specimens to reduce friction and facilitate full swelling of the soil specimens, as well as to seal the sides of specimens used in 1-dimensional flow tests Snyder (2015). However, it was considered that this may be a source of variability. This is because it was also possible that excess amounts of vacuum grease may intermix with soil during its compaction or trimming and affect the inundation and swelling around the edges of the specimen. To evaluate this possibility, a series of specimens of natural (i.e., non-lime treated) Eagle Ford Clay were compacted in clean aluminum testing rings using 1) no vacuum grease, 2) a “typical” amount of vacuum grease (0.05 g), and 3) an “excessive” amount of vacuum grease (0.1 g). The amount of vacuum grease was determined by weighing the aluminum ring before and after coating the inside with vacuum grease. Identical soil specimens were then tested at a target effective stress level of 235 psf. The range of initial moisture content and initial dry density achieved during compaction for the specimens prepared for this testing series is provided in Table 3.2.1, and the swelling data is presented in Figure 3.2.1.

Table 3.2.1: Range of actual values of initial conditions for specimens in test series to assess the effect of using vacuum grease.

	Gravimetric Moisture Content (%)	Void Ratio at Compaction	Dry Density [pcf]
Minimum Value	0.232	0.831	93.9
Maximum Value	0.236	0.847	95.1
Percent Error	1.8%	1.3%	1.9%

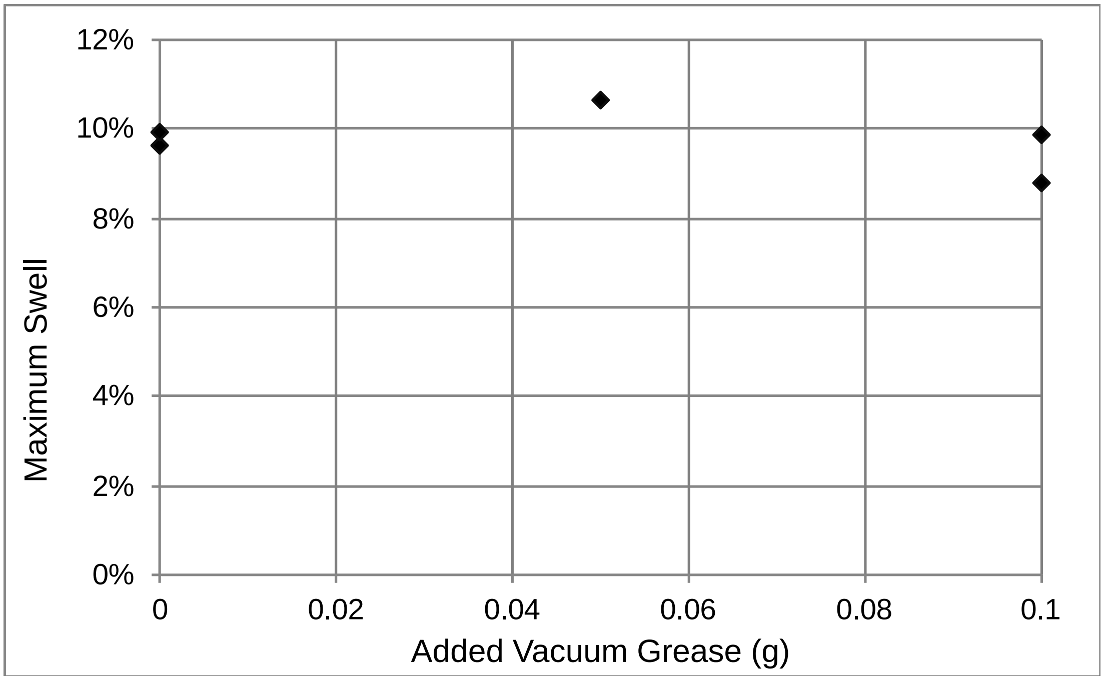


Figure 3.2.1: Variability in maximum swell with vacuum grease used in centrifuge testing of untreated Eagle Ford Clay.

As shown in Figure 3.2.1, the swelling strain values among the 5 specimens vary by about 1.8% (8.8% to 10.6%), which is within the level of scatter observed in routine testing on identical specimens, and consistent with the differences between target and actual initial moisture content and initial dry density in soil specimens. However, it can be observed that the specimens tested with an “excessive” amount of vacuum grease swell somewhat less and present a larger variability than specimens tested without vacuum grease. Additionally, the specimen tested with a “typical” amount of vacuum grease swelled more than the others. Based on these results, it is recommended to use only a very thin coating of vacuum grease on the cutting rings during testing.

3.2.2. Effect of Oven-drying Soil Samples

While it has been the preferred practice to perform swelling tests using soil samples that have been air-dried prior to moisture conditioning, it may occasionally be necessary to have them oven-dried to expedite the soil drying process. However, drying soils at high temperatures may irreversibly affect the mineral composition. This is particularly important in soils with a high organic content, but it can also affect highly expansive clays, as the removal of adsorbed water molecules from clay particles may occur at comparatively high drying temperatures. This loss in bound water has been reported to markedly decrease the soil plasticity and potential to swell (Basma et al. 1995).

Consequently, a test series was performed on Eagle Ford clay to evaluate differences in swelling that may result after testing either air-dried or oven-dried soil samples. The soil was processed and specimens were then prepared at a target initial moisture content of 24% and a target dry unit weight of 95 pcf. Tests were conducted at a target effective stress of 200 psf. Table 3.2.2 provides the range of initial compaction conditions for these specimens.

Table 3.2.2: Range of initial conditions for air-dried and oven-dried samples.

	Moisture Content	Compaction Void Ratio	Dry Density [pcf]
Minimum Value	0.234	0.836	92.8
Maximum Value	0.248	0.849	93.6
Percent Error	6.2%	0.9%	1.6%

Figure 3.2.2 shows the measured swelling strain plotted against the effective stress. The swell in oven-dried specimens swell is 1.3% less, on average, than that in air-dried specimens. However, the variability among test results is essentially the same in each case (a difference of 1.4% in swell for otherwise identically compacted samples).

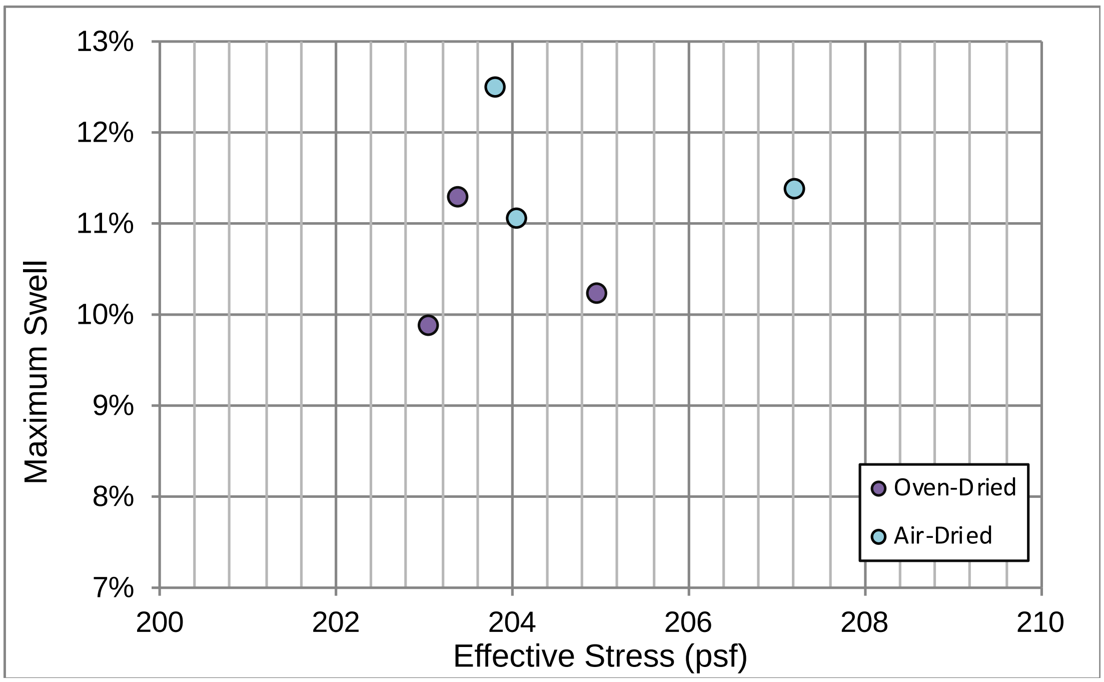


Figure 3.2.2: Variation of maximum swell for untreated air-dried and oven-dried Eagle Ford Clay.

To verify these results with regard to the initial moisture content, Figure 3.2.3 shows the swelling as a function of the initial moisture content. The oven-dried samples were compacted to an initial moisture content that was slightly lower than the air-dried samples (an average initial moisture content of 23.6% for oven-dried versus an average initial moisture content of 24.7% for air-dried). Comparatively drier specimens are expected to show higher swell, as discussed in Section 3.2.2, so the process of oven-drying may have an even more significant impact on the swelling than that reflected in the results plotted in Figure 3.2.2.

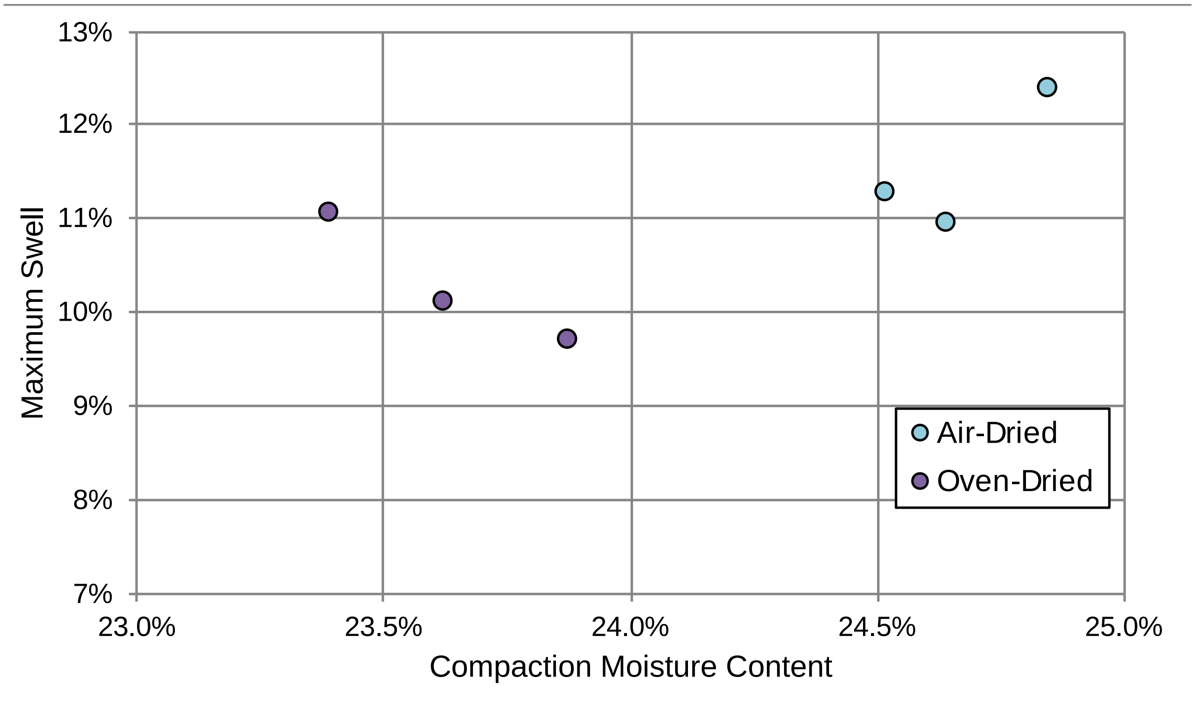


Figure 3.2.3: Variation of maximum swell with initial moisture content for untreated Eagle Ford Clay at 200 psf.

3.2.3. Effect of the Amount of Ponded Water

In addition to the particular specimen preparation details discussed in Section xxx, the amount of ponded water added to each test at the time of initiating infiltration and swell processes was also evaluated as a potential source of variability in the test results. The amount of ponded water can affect the test in two ways: (1) the level of ponded water over the sample will impact the buoyant force acting on the contact displacement sensors (affecting the corresponding overburden load), and (2) if the amount of ponded water is exhausted during the test, the swelling process will be interrupted, and complete swelling may not be achieved. To assess the possible effect of this, samples of air-dried processed Eagle Ford clay were prepared to a target moisture content of 24% and a target dry density of 95 psf and centrifuge tested under an effective stress of approximately 165 psf. Specimens were then inundated with 50, 60, or 100 milliliters of water and allowed to swell. Table 3.2.3 provides the range of initial compaction conditions for this test.

Table 3.2.3: Range of initial compaction conditions for test data shown in Figure 3.2.4.

	Moisture Content	Compaction Void Ratio	Dry Density [pcf]
Minimum Value	0.234	0.818	92.5
Maximum Value	0.247	0.861	94.5
Percent Error	5.6%	5.2%	2.2%

The test results are shown in Figure 3.2.4. The overall difference in swelling among the tests is about 3.2%, which is greater than the combined differences in initial moisture and density.

More specifically, the specimens inundated with 100 mL of water tended to swell a greater amount and more consistently than specimens inundated under 50 or60 mL of water. Additionally, while the specimens tested with 60 mL of water swelled less than the other specimens, they also had a higher initial moisture content by 1% than the other four specimens.

This slight positive trend in swelling with the volume of ponded water is probably not linked to the buoyant force acting on the contact-style sensor used in the centrifuge test setup, as shown in Figure 3.2.5.

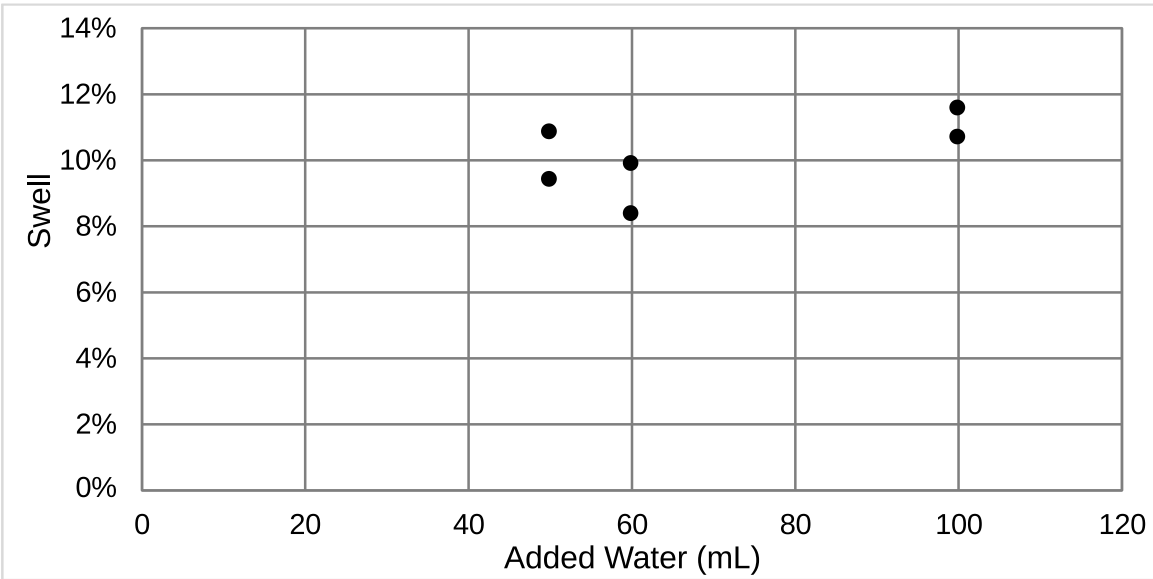


Figure 3.2.4: Variation of swell with amount of water added for untreated Eagle Ford Clay.

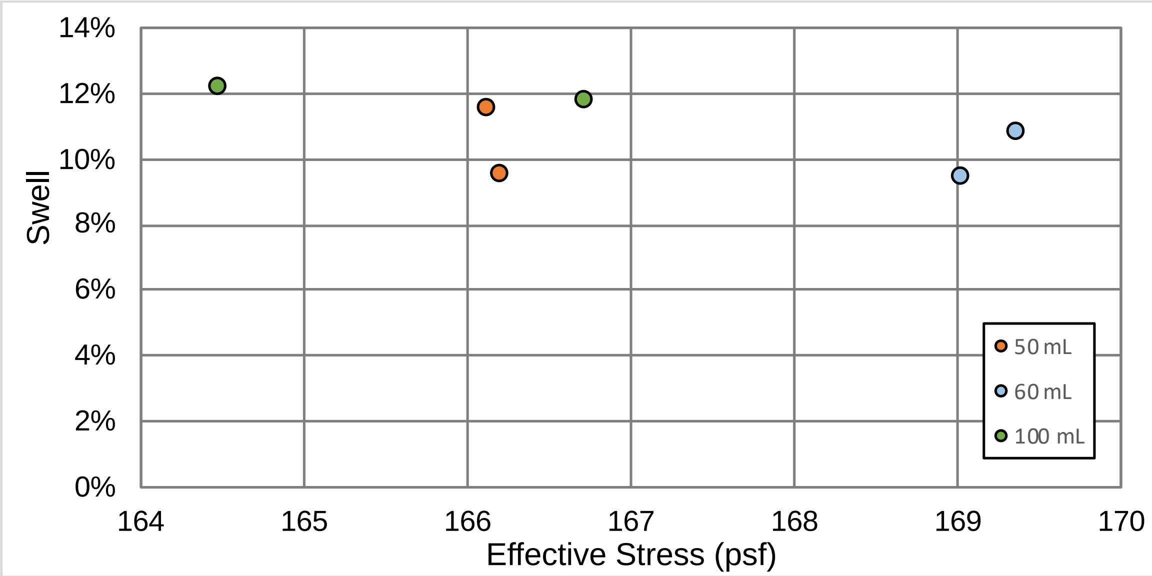


Figure 3.2.5: Swell vs stress for samples inundated with varying amounts of water.

The results of this evaluation have two implications: first, that specimens tested using the same amount of ponded water (within 5 mL) will not likely see a marked variation in swell due to the amount of water added; but second, that the height of water used in the test (and hence the buoyant force upon the position sensor) should be calibrated to the volume added to the test setup.

Consequently, the amount of ponded water in the revised testing protocols is prescribed to be within 5 mL of the maximum fill level for the specific permeameter cup geometry.

3.2.4. The Specified Thickness of the Soil Specimen

Soil specimens confined by a testing ring experience sidewall friction during swelling, and

If the sidewall surface area to volume ratio (reducing to a height-to-diameter ratio for cylindrical specimens) is comparatively large, the swelling in the specimen may not represent free-field conditions accurately. To evaluate this effect, several tests were performed on identical specimens with varying thicknesses.

Figure 3.2.6 shows the effect of specimen thickness upon the final void ratio after swelling, while Figure 3.2.7 shows the ratio between the strain and the ultimate strain as a function of time for the same three specimens. This data shows that increasing the specimen thickness primarily extends the testing time, although a slight negative trend in swelling may be evident in the data for thicker specimens. Consequently, it is considered that a 1 cm specimen is appropriate and convenient for testing the expansiveness of most soils, but taller specimens are acceptable for cases in which the specimen thickness cannot be prescribed, such as in field-trimmed samples or samples with comparatively large particles.

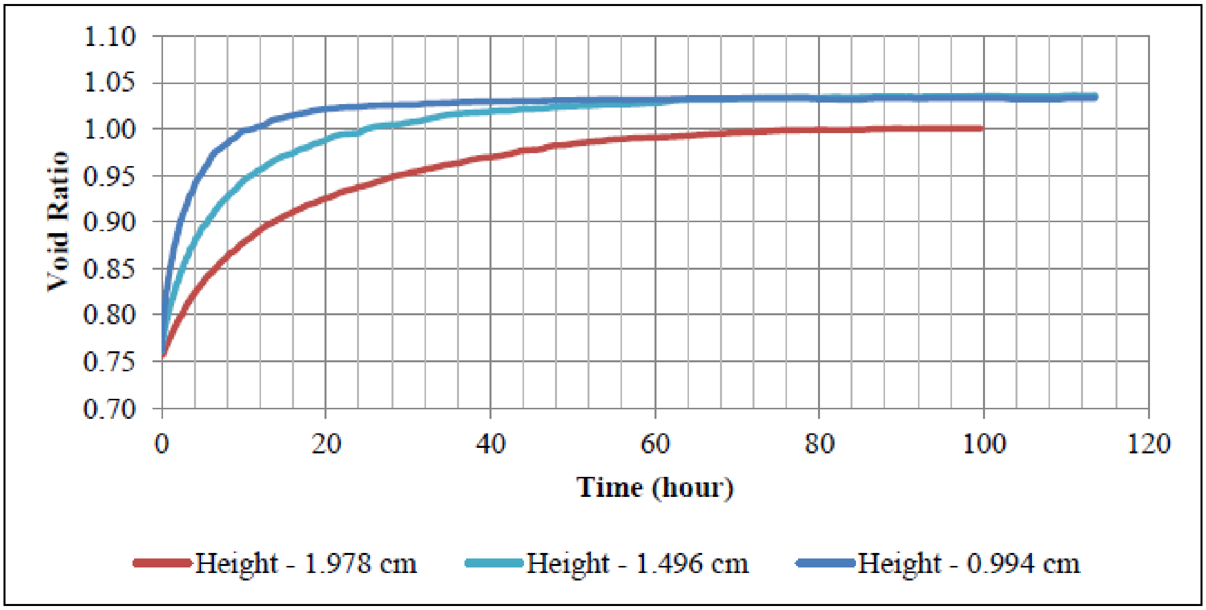


Figure 3.2.6: Void ratio vs time for specimens of varying thicknesses (Armstrong, 2018).

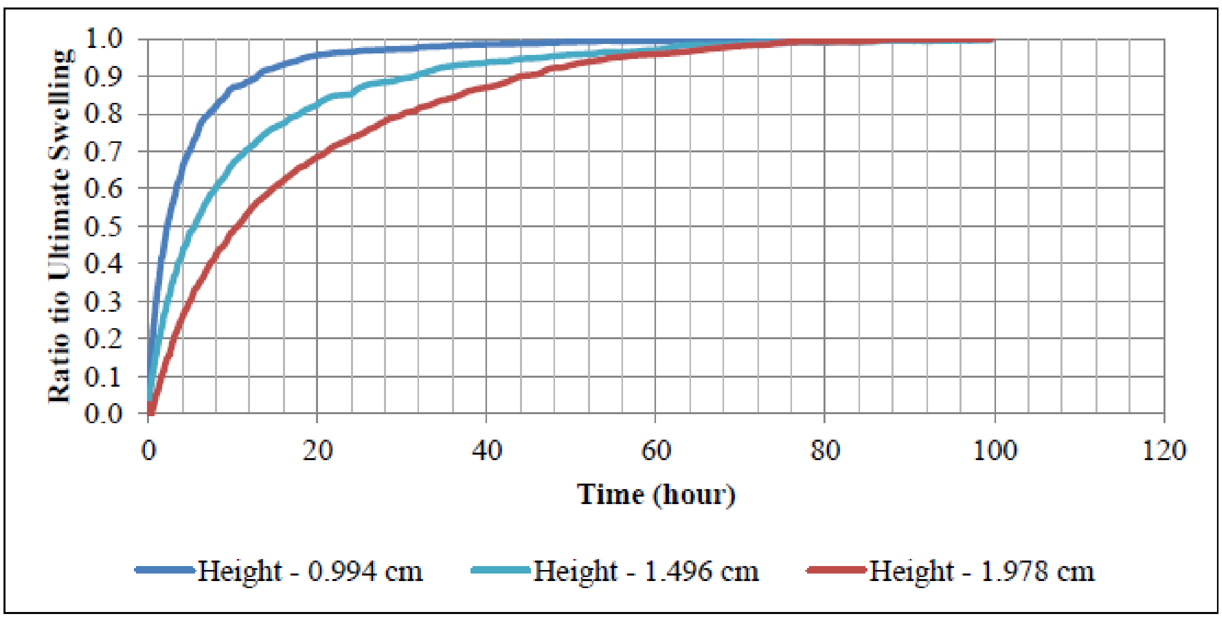


Figure 3.2.7: Ratio to ultimate swelling vs time for Eagle Ford Clay specimens, showing increased time to end of primary swell with increased specimen thickness (Armstrong 2018).

3.2.5. Effect of Local Variations in the Initial Specimen Thickness

The effect of local variations in the initial specimen thickness during their preparation was also evaluated. The testing protocols indicate that soil specimens should be compacted to an average target thickness of 1.00 cm, or 0.393 inches, and that this should be verified at the center of the specimen and at 4 points around the specimen edge, as shown in Figure 3.2.8:

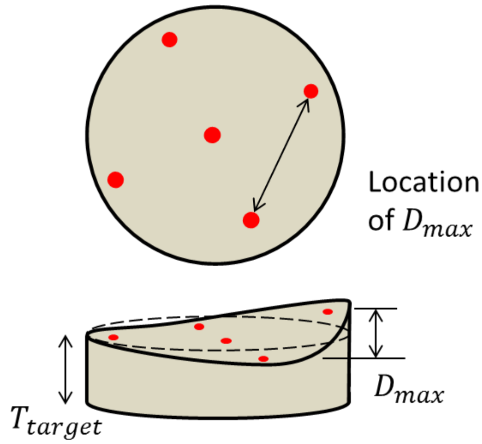


Figure 3.2.8: Diagram showing specimen thickness measurements and deviations.

A variation of +/- 0.005 inches (+/- 1.2%) is considered acceptable to accurately define the initial thickness and volume of the specimen for use in subsequent calculations. Because displacements during the centrifuge test are measured with a sensor resting on a rigid porous disc atop the soil specimen, local variations of the specimen thickness would affect possibly significantly the interpretation of the measured displacements.

To assess this effect, a subset of the untreated Eagle Ford data was analyzed for the maximum local deviation in thickness, referred to in Figure 3.2.8 as D_{max}. Data from this analysis was compared with the aggregated swell-stress data from Eagle Ford specimens prepared under the same global conditions.
The initial conditions for this data are shown in Table 3.2.4.

The data was then binned according to the value of D_{max} for each specimen. Values of D_{max} ranged from 4 thousandths of an inch (mils) to 13 thousandths of an inch. Figure 3.2.9 shows the swelling results within each category. For clarity in the results, the swelling data has been model-adjusted to an initial moisture content of 24.5%, as outlined in Section 2.2.2.

Table 3.2.4: Range of initial conditions for evaluation of local thickness variation.

	Moisture Content	Dry Density [pcf]
Minimum Value	0.15	80
Maximum Value	0.35	108
Model-adjusted Value	0.245	94

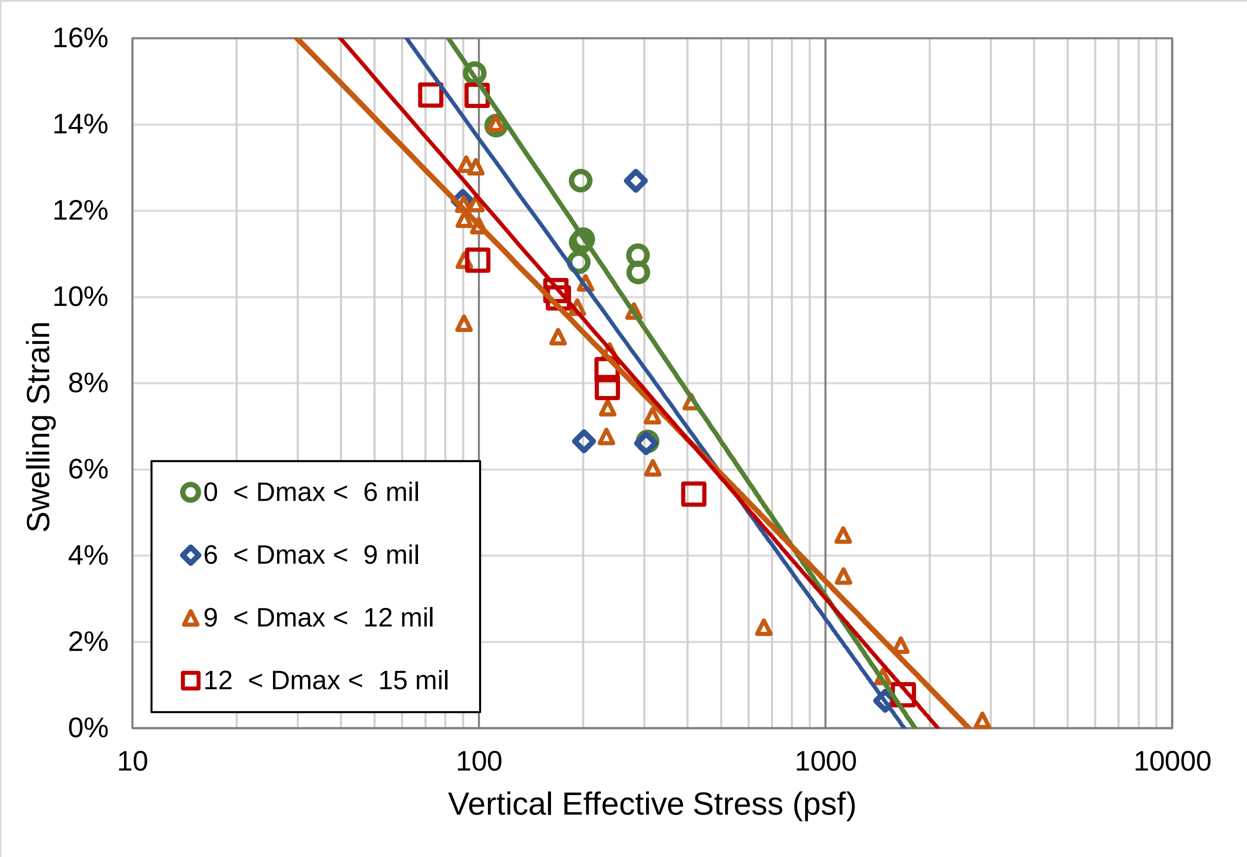


Figure 3.2.9: Model-adjusted swell-stress data grouped by maximum local variation in thickness.

The results in the figure show that the magnitude of the maximum differential height does not significantly affect the magnitude of swell or the amount of variation in the data. This may indicate that a maximum differential height of 0.013 inches (a difference in thickness of 3.3% for a 0.393-inch specimen) is not enough to significantly affect the variability in swelling. Currently, however, it is still recommended in the testing protocols that the differential height across the specimen be minimized to preferably less than 0.01 inches, so as to minimize error in the computation of volume and density.

3.2.6. Effect of the Approach Adopted to Achieve the Target Vertical Effective Stress

A critical parameter governing the swelling in a particular soil is the average effective stress acting on the specimen. In centrifuge testing, vertical loads are generated in the specimen by radial centrifugal forces according to the equation:

$$\sigma = \int_{r_0}^{r} (\rho \, r \, \omega^2) \, dr \tag{3.1}$$

Where

- σ is the radial stress (equal to the radial force divided by the area of the specimen),
- ρ is the mass density,
- r is the radius of operation,
- r_0 is the reference position within a specimen, and
- ω is the angular frequency (often expressed in revolutions per minute, RPM)

This equation, when integrated for constant density with position, r, gives:

$$\sigma = \frac{\rho}{2}\omega^2(r^2 - r_0^2) \tag{3.2}$$

Consequently, the vertical (radial) effective stresses, σ', acting on a centrifuge specimen should be equal to:

$$\sigma' = \sigma - u = \frac{\omega^2}{2}\left(\rho_{total}(r^2 - r_0^2) - \rho_{water}\left(r^2 - r_{0,w}^2\right)\right) \tag{3.3}$$

Where u is the pore pressure under ponded water conditions, derived in the same manner above

This equation is illustrated in Figure 3.2.10, in which the specimen additionally feels the stress contribution from a brass disk placed as overburden on top of the specimen. In this case, the integration of Equation 3.1 becomes the sum of the contributions from each component, computed using Equation 3.2 within each material domain. In the figure, the specimen is shown with the radial direction in a vertical orientation to emphasize the equivalence between the state of stresses in the field and in a centrifuge test.

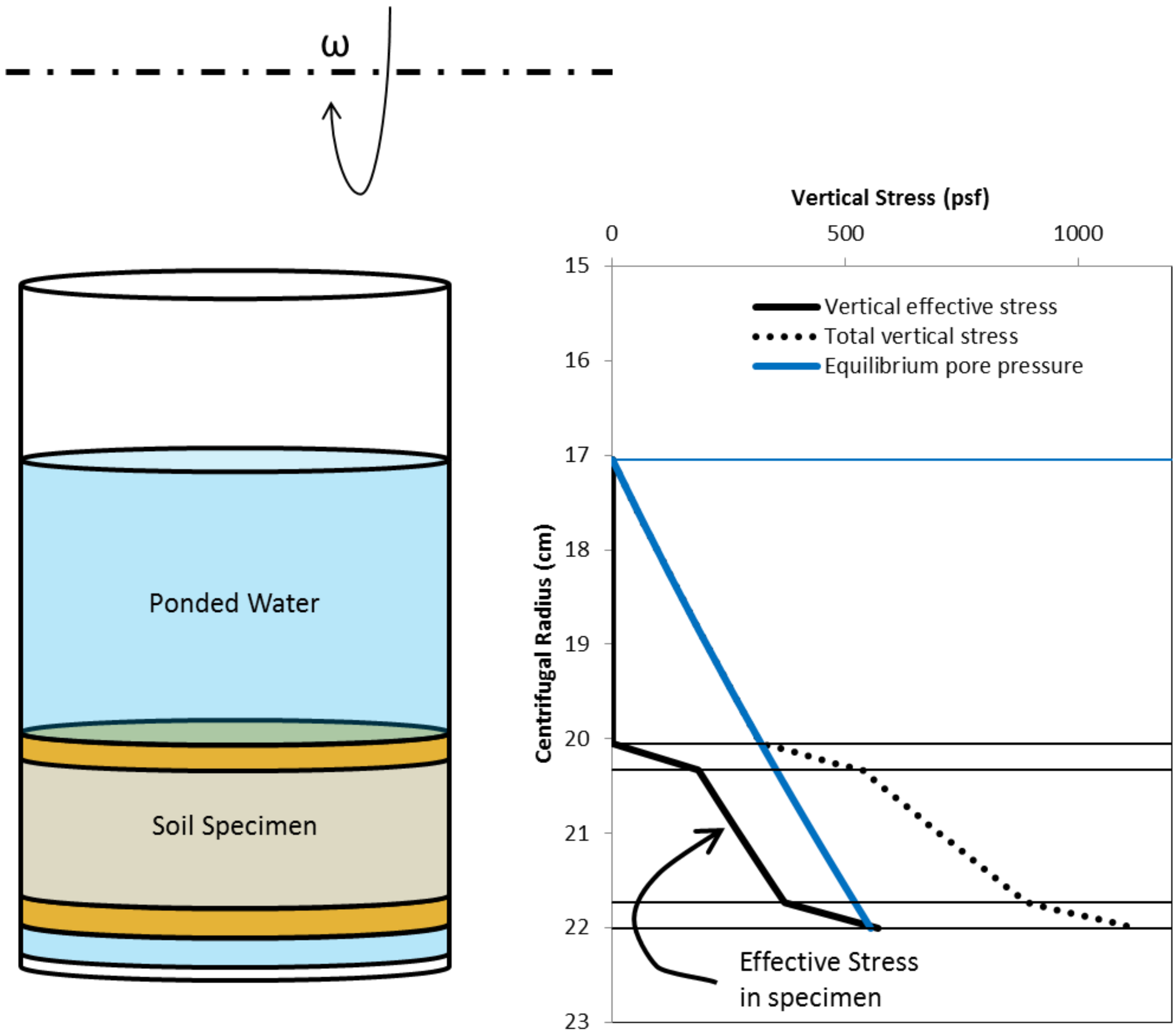


Figure 3.2.10: Generation of stress in a centrifuge specimen.

Because the radial stress in a centrifuge is proportional to the rotational radius of operation, and because the specimens may have significant thickness in relation to the radius of operation, the stresses at the top of the specimen can be substantially lower than those at the bottom of a specimen. Consequently, the use of a larger overburden mass will produce a more uniform stress distribution throughout the specimen compared to the use of a larger G-level with a smaller overburden mass, as shown in Figure 3.2.11.

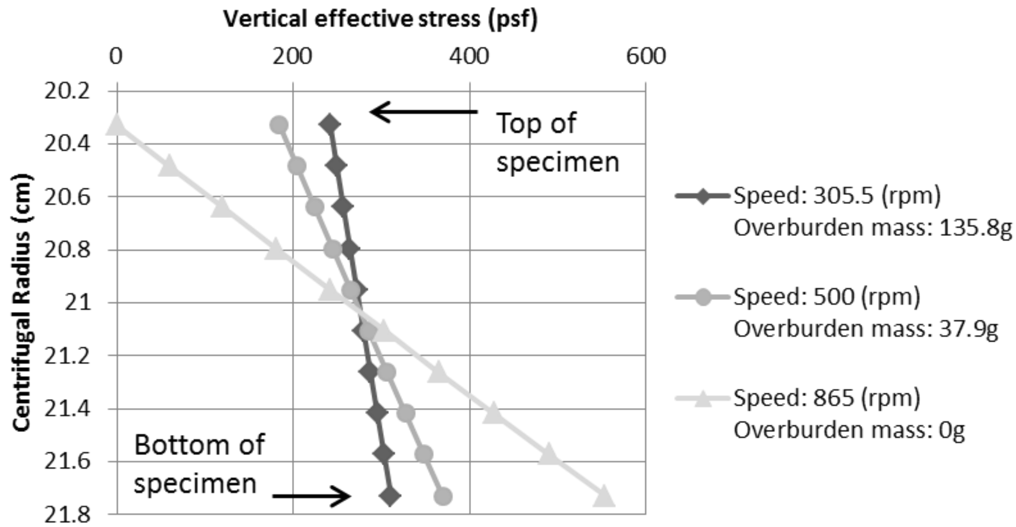


Figure 3.2.11: Equivalence of effective stress generated by different combinations of overburden load and g-level.

Mathematically, the use of the equivalent stress framework developed by Zornberg et al. (2013) accounts for this effect. As a consequence, the various approaches to generate a target vertical effective stress in the specimen should produce equivalent swelling strains. Figure 3.2.12 shows the results from four test series performed by Kuhn (2010), demonstrating the similarity in methods of applying the vertical load to the specimen. In these tests, Series (i) used a 1-D oedometer load to apply the stress, while Series (ii – iv) used the large permeameter centrifuge located at the University of Texas at Austin to apply vertical stresses using a combination of surcharge mass and centrifugal acceleration.

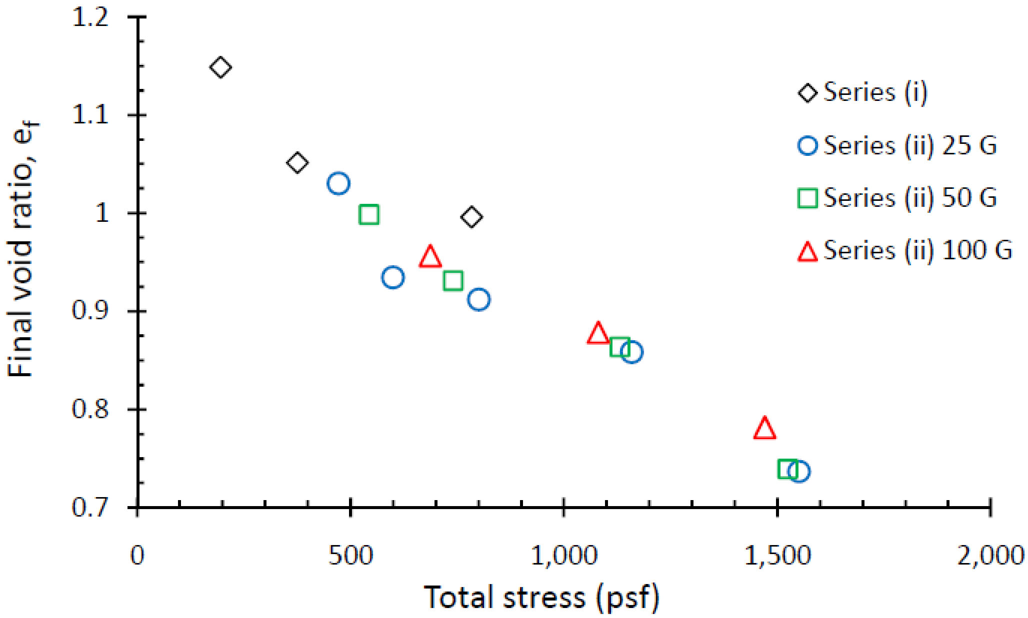


Figure 3.2.12: Trends in void ratio vs total stress, showing equivalence between g-level and overburden mass (Kuhn, 2010).

Chapter 4. Incorporation of Centrifuge Testing into PVR Methodology and Design

Potential vertical rise is a concept based on a certain ‘active zone’ within a soil profile (usually thought to correspond to a depth of 10 to 15 feet in Central Texas) going from an initial uniformly dry state to a saturated state. The resulting changes in void ratio associated with the ingress of moisture lead to vertical strains (and lateral pressures) which cause a net heave at the ground surface. Consequently, if the swelling vs effective stress curves are known for a given soil profile, these curves can be integrated with respect to depth to give an estimate for the potential heave at the ground surface.

TxDOT method TEX-124-E makes use of this principle to provide a correlation between soil index parameters and expected swelling of the soil. In the Texas Swell Testing method (centrifuge testing method), the primary improvement is that swelling is measured for a given set of conditions, rather than inferred from a correlation.

In addition, swelling in the presence of a chemical additive can be directly measured using this test method.

From a practical standpoint, the slope of the swell-stress line can usually be adjusted to fit the swelling measured on chemically treated samples at a low stress, by assuming a constant swell pressure with treatment dosage (which may be statistically appropriate in many cases, as shown in Section 2.3). Because of scatter in real testing data, it is recommended to have at least one repeat specimen of each chemical dosage.

4.1. Methods of Curve-fitting

The direct method of determining the PVR of a soil horizon using centrifuge technology has been in development since originally documented by Snyder (2015). This method is referred to as the DMS-C approach, or Direct Measurement of Swelling using Centrifuge technology. Data from the linear position sensors in contact with swelling clay specimens is used to produce a swell-time curve for each centrifuge test specimen. The swelling strain determined from the end of the primary swelling phase is then plotted against the effective stress acting on the specimen, as examined in detail in Chapters 2 and 3. This stress is dependent on the centrifuge g-level, the weight of the overburden disc on the specimen, and the amount of water added to the specimen as discussed in Section 3.2.6. Tests were performed at 3 g-levels to produce swell data for a range of effective stresses between approximately 100 psf and 1000 psf.

A curve fitting function was developed to apply to the swell-stress data, which could then be numerically integrated to calculate the PVR of the soil stratum. The curve fitting function, shown in Equation 5.1, is based on a model developed by Plaisted (2015), and requires 3 fitting parameters. Parameter A represents the “free swell” of the soil, or the swell measured at 1 kPa. Parameter B represents the minimum swell of the soil. Parameter C is a curve-fitting variable; after analysis of several values of C, it was determined that a value of 60 produced the best fit curve. The final equation used is shown in Equation 4.2 and requires 2 parameters. Solving for the curve-fitting parameters A and B can be accomplished by using the Solver function in Microsoft Excel to minimize the root mean square error (RMSE) of the function (Equation 4.3). An example curve is shown in Figure 4.1.1.

$$\epsilon(\sigma') = \frac{(A-B)\ln(0.01C+e)}{\ln\left(C\frac{\sigma'}{\sigma'_{ATM}}+e\right)} + B \qquad Eq. 4.1$$

$$\epsilon(\sigma') = \frac{(A-B)*1.197}{\ln\left(60\frac{\sigma'}{\sigma'_{ATM}}+e\right)} + B \qquad Eq. 4.2$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}\left(\epsilon_{measured,i} - \epsilon_{calculated,i}\right)^2}{n}} \qquad Eq. 4.3$$

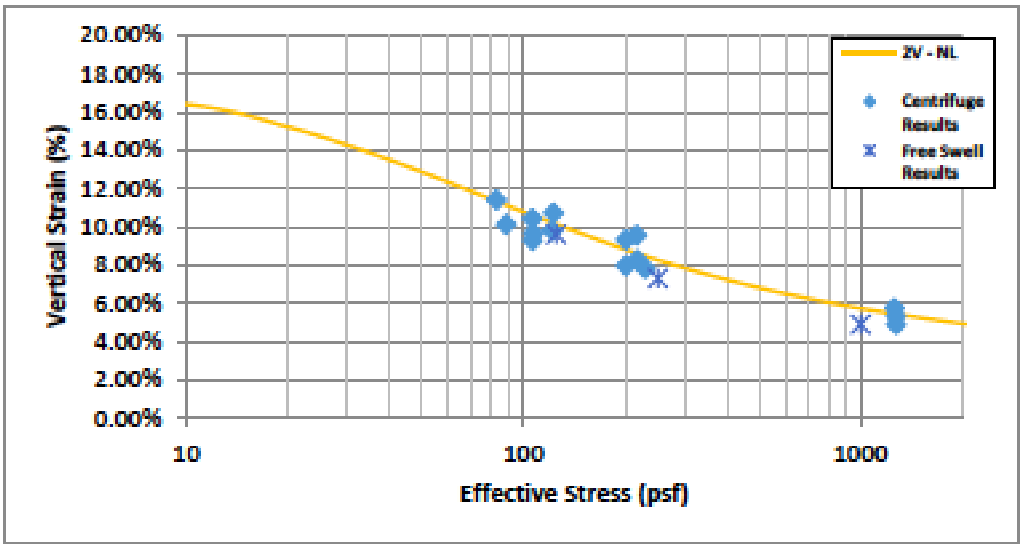


Figure 4.1.1: Example of equation 4.2 fit to swell-stress data (Snyder 2015).

The curve fitting function was later adjusted, and another variable was added, resulting in Equation 4.4. Here, the A parameter continues to represent the "free swell" value, and D is a curve-fitting parameter that is taken to be 60. The B and C parameter differ, however—here B affects the curvature of the inflection point, and C affects the effective stress at which the inflection point of the curve occurs. The equation is solved in the same way as the previous iteration—via the Solver function in Excel. The parameters A, B, and C are adjusted to minimize the RMSE of the curve fitting equation. An example of the curve fit to swell-stress data is shown in Figure 4.1.2.

$$\epsilon(\sigma') = A \frac{\ln\left[\left(\frac{D * 20.89}{C * \sigma'_{ATM}}\right)^{B\left(1+\left(\frac{20.89}{C*\sigma'_{ATM}}\right)^{1.5}\right)^{8}} + e\right]}{\ln\left[\left(\frac{D * \sigma'}{C * \sigma'_{ATM}}\right)^{B\left(1+\left(\frac{\sigma'}{C*\sigma'_{ATM}}\right)^{1.5}\right)^{8}} + e\right]} \qquad Eq. 4.4$$

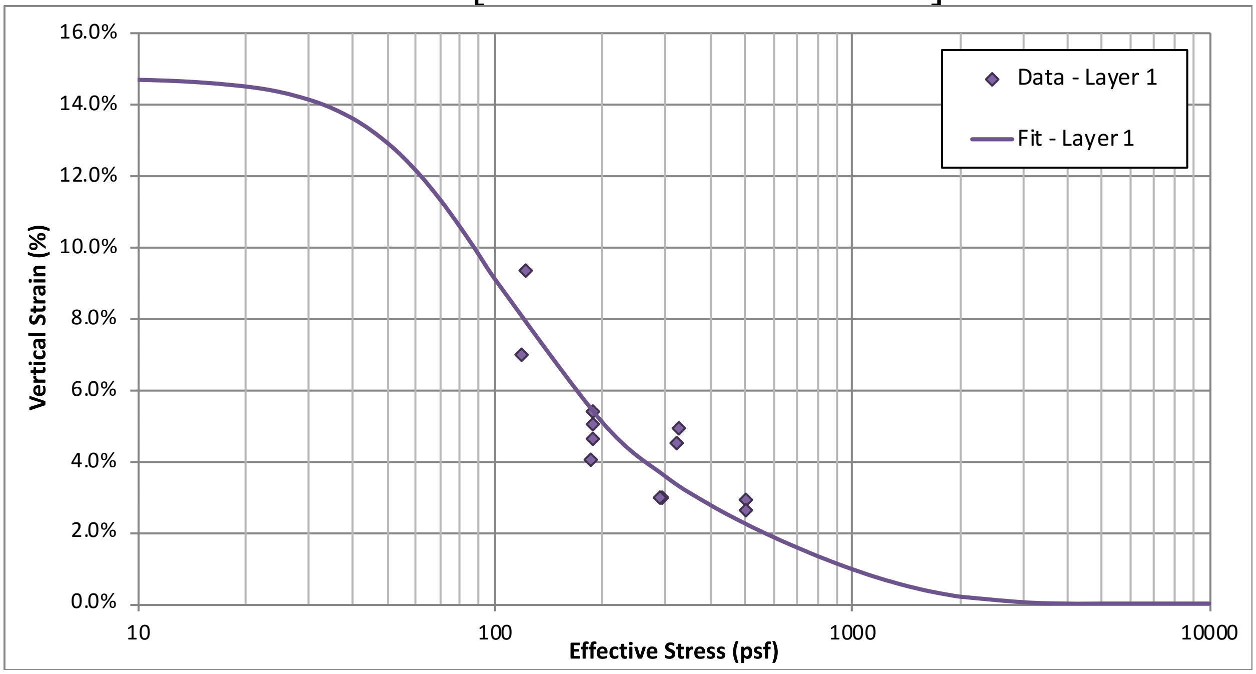


Figure 4.1.2: Example of equation 4.4 fit to swell-stress data.

These curve fitting functions have the benefit of being a fairly accurate representation of site-specific swell-stress data. However, the curves generally require at least 3-4 distinct data points (or at least one full centrifuge test) per soil layer. Particularly when a lime-treated PVR estimate is desired and several lime dosages must be tested for each soil, the number of required tests may become significantly prohibitive. Thus, a reduced procedure for producing site-specific untreated and lime-treated swell-stress curves is desired.

4.2. Reduced Testing Method Utilizing Log-Linear Functions

It can be seen for both curve-fitting functions discussed in Section 2.3, that the swell-stress curve very closely approximates a straight line in semi-log space within an effective stress range of 100 psf to 1000 psf, which is the general stress range of interest for PVR calculations in typical soil strata. Equation 4.5 is the equation of this line, where parameters A and B are the slope and intercept (at a stress value of unity) of the line, respectively.

$$\epsilon(\sigma) = A * \log \sigma + B \qquad Eq. 4.5$$

An example of swell-stress data fit to a log-linear function is shown in Figure 4.2.1. The orange line denotes untreated expansive clay, while the purple and green lines denote expansive clay treated with 1% and 2% hydrated lime, respectively. While each data set closely follows a log-linear swell-stress relationship, the point at which each line intersects the x-axis, the swell pressure, varies.

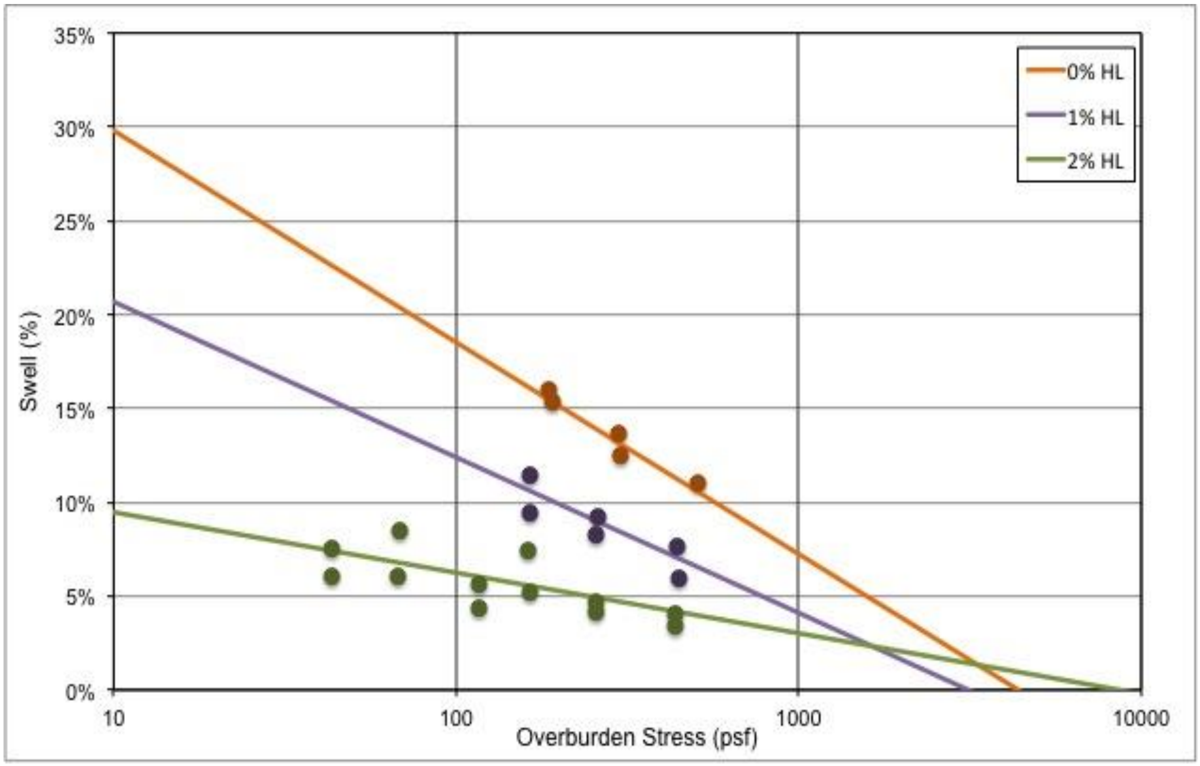


Figure 4.2.1: Example swell-stress data for untreated and lime-treated expansive clay.

While it is understood that the concept of a single unique swell pressure for a particular soil is an assumption of the method, the model simplification using a constant swell pressure may allow for the development of an optimized testing procedure for both untreated and lime-treated expansive clays. Figure 4.2.2 (a) illustrates the idealization of the plots in Figure 4.2.1 where all three swell-stress curves converge to a single swell pressure. The goal is that, for an effective stress range of approximately 100 psf to 1000 psf, the variation in lime-treated swell-stress curves from a best-fit line to a line of constant swell pressure will result in a reasonably similar line. Moreover, the area under the lines plotted in Figure 4.2.2 (a) are to be reasonably close to the area under the lines plotted in Figure 4.2.1, thus resulting in the same or nearly-the-same calculations of PVR.

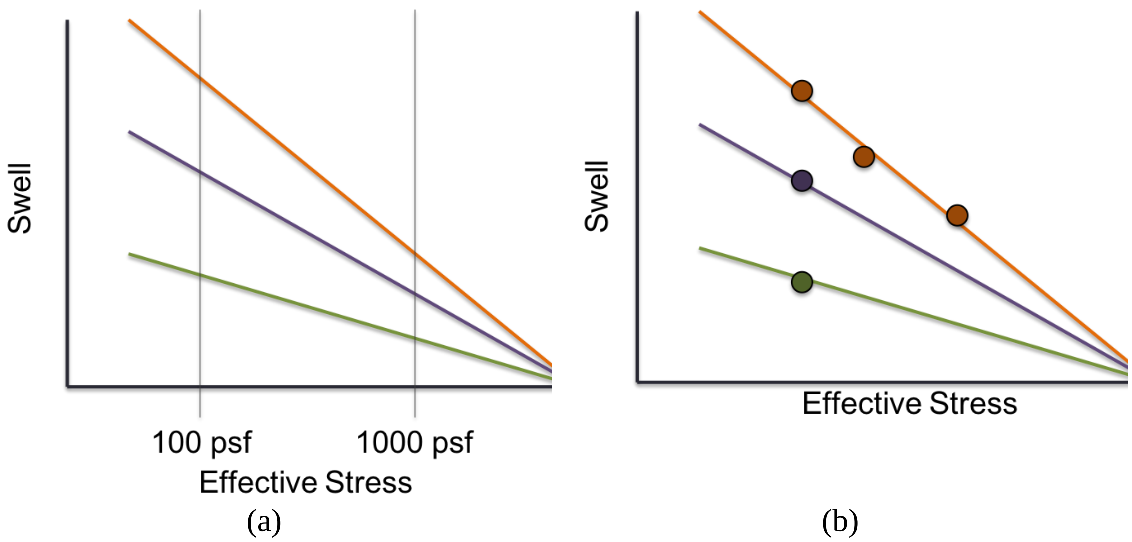


Figure 4.2.2: (a) Example of converging swell-stress curves; and (b) example of reduced method of producing swell-stress curve family.

Figure 4.2.2 (b) illustrates the plan for a Reduced Method of producing a lime-treated swell-stress curve family for a given soil. In this scheme, the full swell-stress curve is generated from the natural soil in a given location, represented by the orange data points, and then representative treated samples are tested in the lowest stress range anticipated, represented here by the purple and green data points. These data point for each lime dosage will be extended to match the extrapolated swell pressure of the untreated soil to create a family of lime-treated swell-stress curves, with which the lime-treated PVR of the soil may be calculated.

Chapter 5. Quantification of PVR for Characterization and Remediation Design in Roadway Projects

5.1. Overview

Eight field sites were selected to quantify the severity of potential problems associated with the presence of expansive clays at those locations. Specifically, soil data was collected and characterized at each location in order to quantify the PVR using the approach involving the use of Texas Swell Tests. The use of lime stabilization is evaluated as possible remediation at some of the locations. Information is provided in this chapter to describe each site as well as the activities performed at each of them.

Table 5.1.1 shows an overview of the different activities conducted at each site. Results from the Total Station and Condition Surveys for Sites 3 and 4 can be found in Zheng (2018). Figure 5.1.1 shows a map with the selected sites, while Figure 5.1.2 to Figure 5.1.4 show these sites grouped according to their respective district within TxDOT.

Table 5.1.1: Selected activities by site.

Site No.	Site	County	District	Condition Survey	Total Station Survey	Subgrade Moisture Monitoring	Subgrade Sample Collection	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization
1	I-35 & Hester's Crossing	Williamson	Austin				✓	✓	✓		✓
2	FM 2	Grimes	Bryan	✓			✓	✓	✓		
3	Turnersville Rd.	Travis	Austin	✓	✓		✓	✓	✓		
4	FM 972	Williamson	Austin	✓	✓		✓	✓	✓		
5	Old Pearsall Rd & Five Palms Dr.	Bexar	San Antonio				✓	✓	✓		
6	US 87 (I-10 to Rigsby Rd)	Bexar	San Antonio				✓	✓	✓	✓	✓
7	US 183 & Martin Luther King Jr. Blvd	Travis	Austin				✓	✓	✓	✓	
8	FM 685	Williamson	Austin	✓	✓	✓	✓	✓	✓		

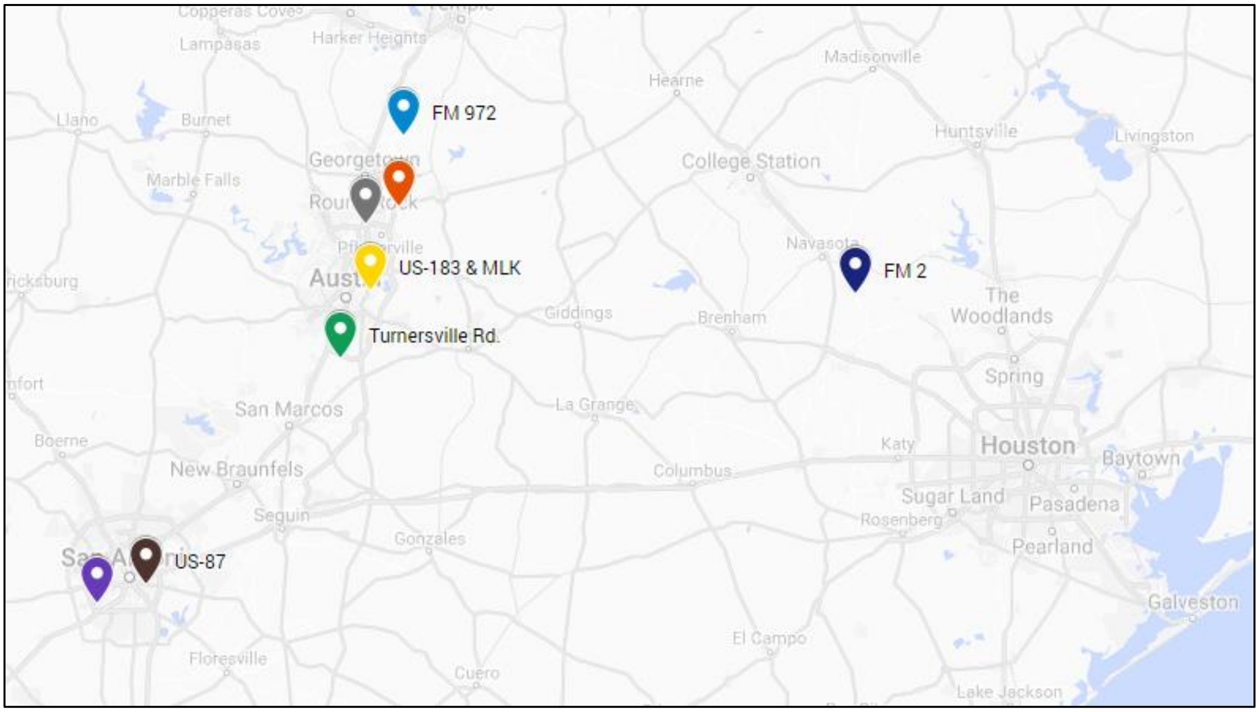


Figure 5.1.1: Map showing general location of selected sites.

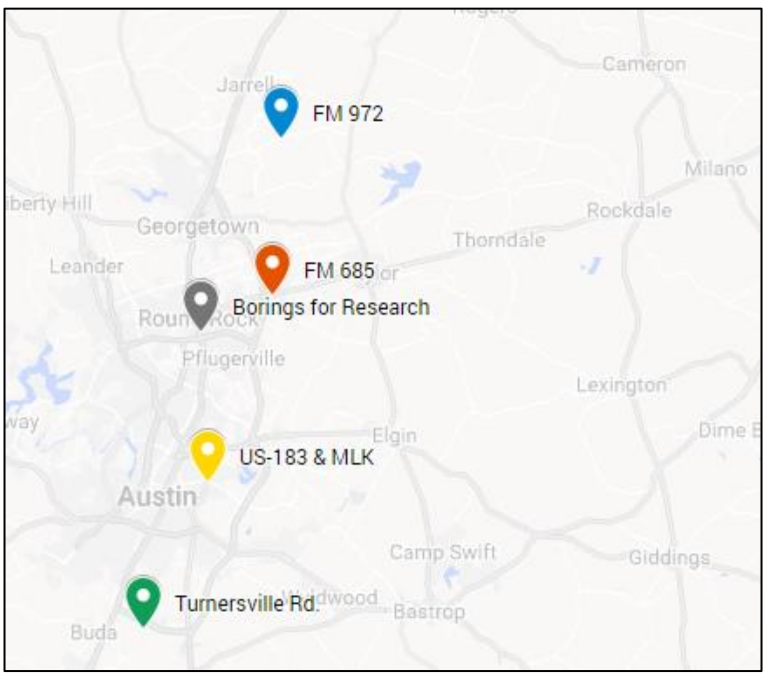


Figure 5.1.2: Map showing general location of selected sites from TxDOT's Austin District.

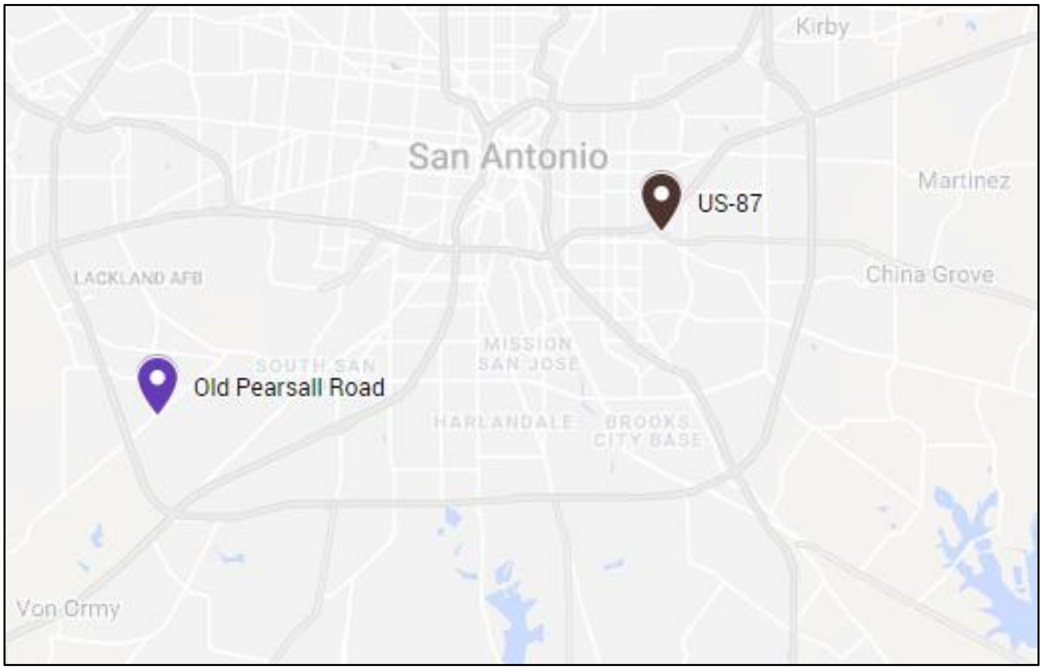


Figure 5.1.3: Selected sites from TxDOT's San Antonio District.

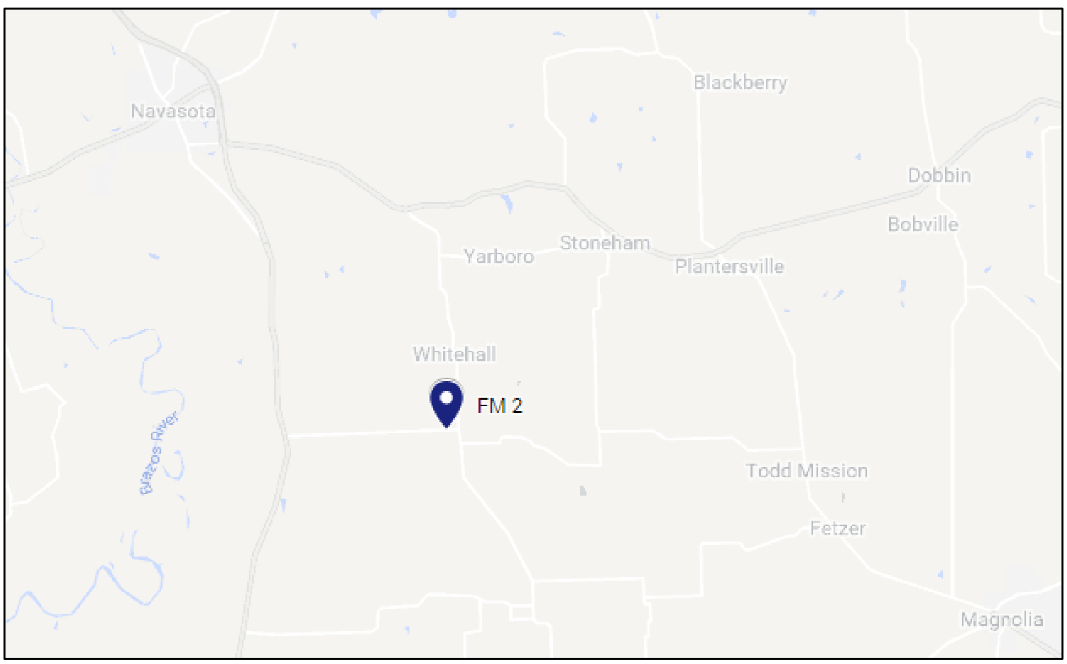


Figure 5.1.4: Selected sites from TxDOT's Bryan District.

The selection criteria for these sites included:

1. Sites in which a history of previous monitoring was available regarding the pavement distress (FM 972, Turnersville Rd, FM 2, Hester's Crossing). For these sites, borings were conducted for soil samples collection and subsequent PVR determination using centrifuge

technology, in order to contribute to assess the suitability of PVR in the context of pavement distress.

2. Sites with planned or ongoing construction activities (US 183, US 87, Old Pearsall Rd, FM 685). These sites were chosen in order to provide support to TxDOT's ongoing planning or construction activities. US 87 and Old Pearsall Rd. were sites targeted for pavement reconstruction, while US 183 was under construction at the time of sampling, and construction at FM 685 offered a good opportunity to install moisture sensors in a pavement subgrade prior to paving at the site.

At each site, and independent of the criteria for their selection, soil conditions were preliminarily evaluated using the United States Department of Agriculture soil survey maps (accessed through the USDA web soil survey). Information from the mapped soil types and from pavement distress was useful to select the location of test sections and borings. At sites with planned or ongoing construction, the site location and boring pattern was based on the actual project needs.

Whenever possible, boring logs are correlated with mapped USDA soil types. At some sites, this was not possible due to the shallow depth of applicability of the USDA maps, and the depth of cut at the site, such as at US 183.

Texas Swell Tests were conducted with the objective of generating the swell-stress curves needed for PVR determination. The tests were conducted, at a minimum, on each unique soil type encountered at each site. In some cases, additional tests were conducted to assess the homogeneity across the site. The details of each PVR calculation are contained in Appendix B.

Thresholds used to categorize the PVR are summarized in Table 5.1.2 modified from Snyder (2015). It should be noted that the threshold between "Moderate" and High" for State Highways and FM Roads correspond to those reported in the TxDOT Pavement Design Manual (2019). Specifically, the TxDOT Pavement Design Manual requires that "the maximum allowable amount of PVR for design is 1.5 in. for main lanes (2.0 in. for frontage roads, when allowed), or less conservative (higher allowable swell) as established by individual district standard operating procedures."

Table 5.1.2: PVR thresholds.

PVR Thresholds	Minimal	Moderate	High	Severe
Interstate Highways	0 – 0.5	0.5 - 1.0	1.0 - 4.0	>4.0
State Highways	0 – 1.0	1.0 - 1.5	1.5 - 4.5	>4.5
FM Roads and Frontage Roads	0 – 1.5	1.5 - 2.0	2.0 - 5.5	>5.5

Texas Swell Tests were conducted using lime stabilized specimens at sites where lime stabilization alternatives were evaluated. The results of these analyses are presented in terms of treatment depths necessary to decrease the PVR corresponding to the original profile (untreated soils) to the maximum PVR value designated as the design criterion.

5.2. I-35 and Hester’s Crossing

This site was selected for sampling based on historical data on the relatively high plasticity and swelling potential of soils encountered in a cut slope above southbound I-35.

Three borings were performed in the vicinity to locate the depth to the highly plastic stratum, identified as the Eagle Ford Clay. Bulk soil samples were subsequently collected to procure enough material for the baseline testing series. Figure 5.2.1 shows the location of sampling. The major findings from the baseline testing series on this material are covered in detail in Section 2.2.

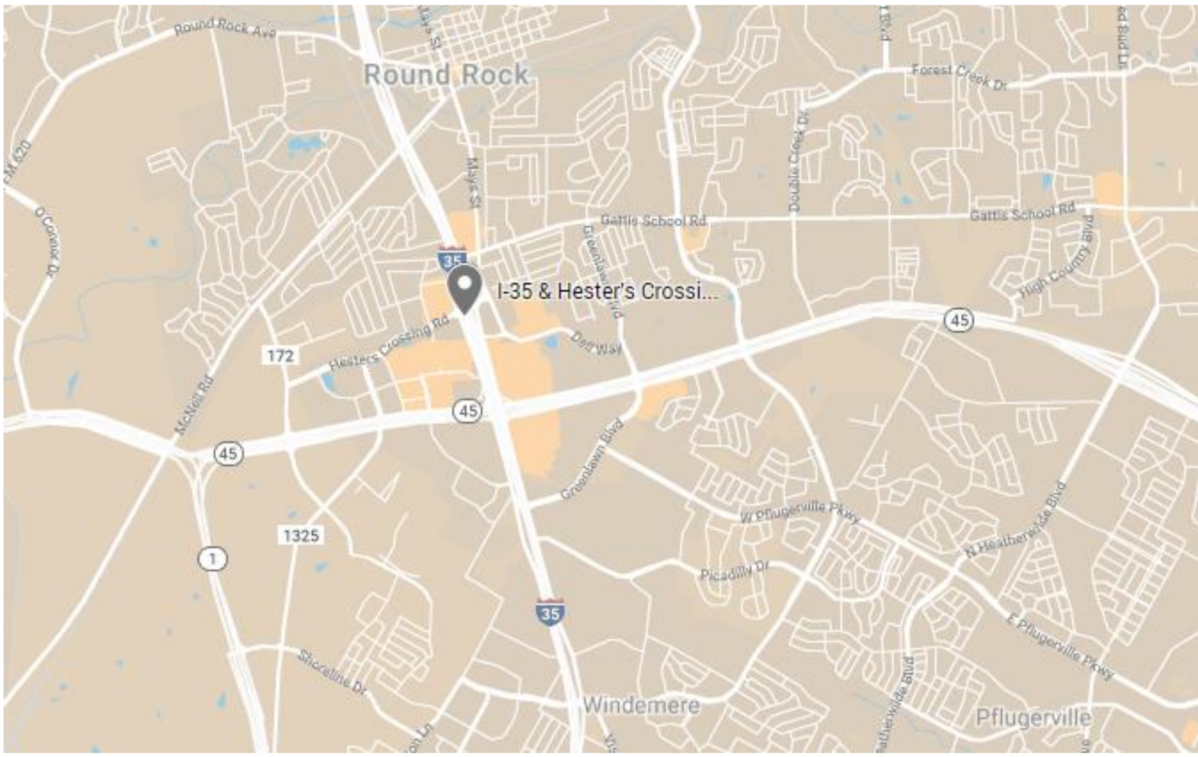


Figure 5.2.1: Location of sampling at I-35 & Hester’s Crossing.

Boring locations were selected with guidance from mapped soil units, which indicated the presence of a highly plastic clay. However, the soils with the highest plasticity were found in Boring HC-1, and probably do not correspond to the soil unit mapped at that location (Austin Series AuB or the Crawford soils CfB). Instead, the material located in the boring most probably corresponds to the Heiden clay (HeC2). Figure 5.2.2 shows the USDA soil survey map shaded according to the average estimated Liquid Limit of each soil type over the top 10 feet of depth.

(a)

(b)

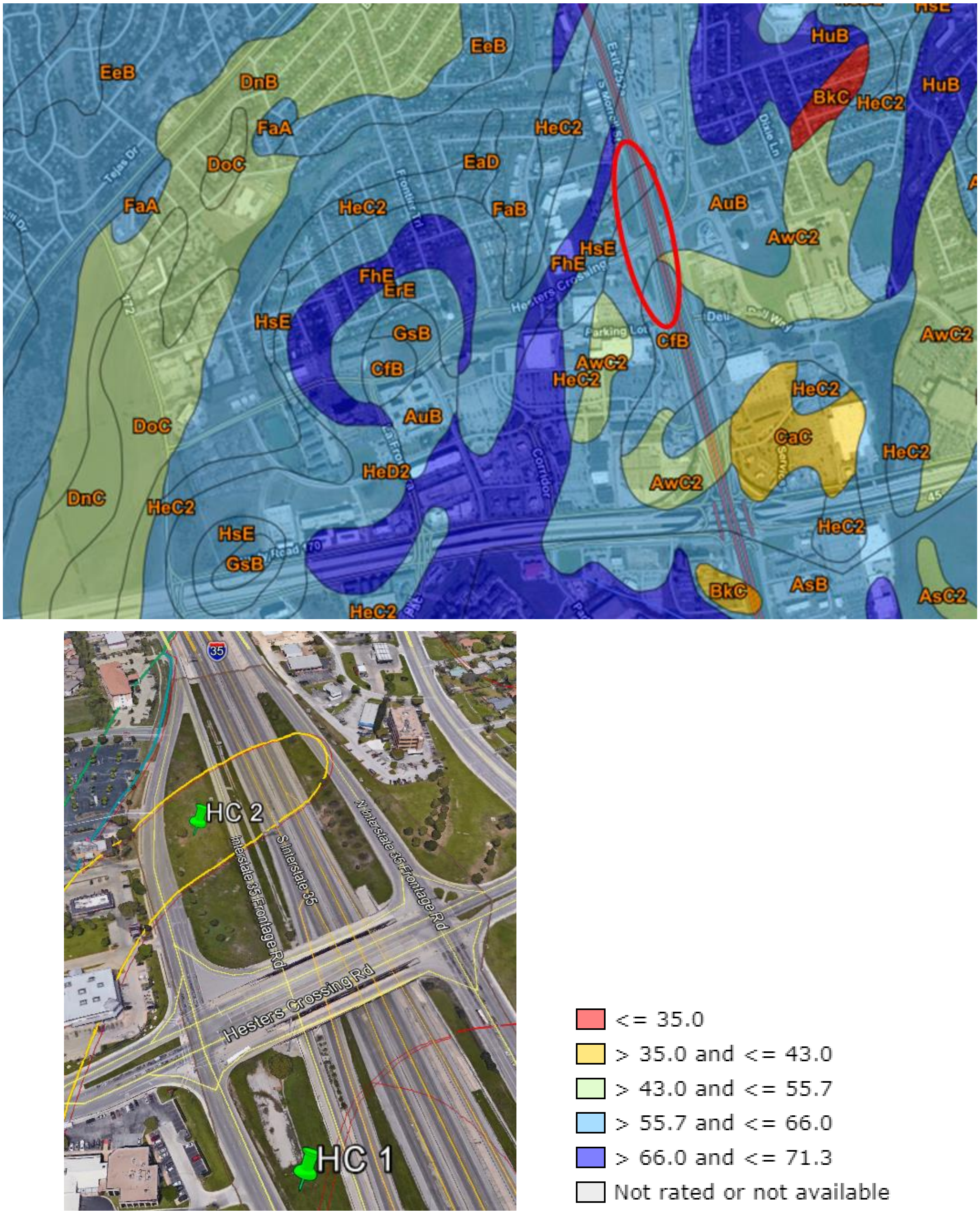


Figure 5.2.2: USDA depth-averaged liquid limit for the top 10 feet at I-35 and Hester's Crossing.

Table 5.2.1 provides a list of the borings conducted at this site, which includes the performed soil characterization tests. In addition, the bulk samples retrieved from this site were used as the primary material in the baseline testing series.

Table 5.2.1: Borings and tests conducted at I-35 & Hester's Crossing.

Boring	Location	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Compaction Testing	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization
HC-1		HeC2	X	X	X	X	X			
HC-1R		HeC2	X	X			X			X
HC-2		HsE	X	X						

The results of the in-situ moisture content and Atterberg Limits tests for the materials in these borings are shown in Figure 5.2.3. Because HC-1R was situated down-slope from HC-1, the results were corrected to the same ground surface elevation as in HC-1. This data shows that a highly plastic shale layer begins roughly 10 feet below the ground surface at the sampling location. While highly plastic clay extends at least an additional 10 feet below the top of the deposit, the highest plasticity clay was encountered at the top of this deposit.

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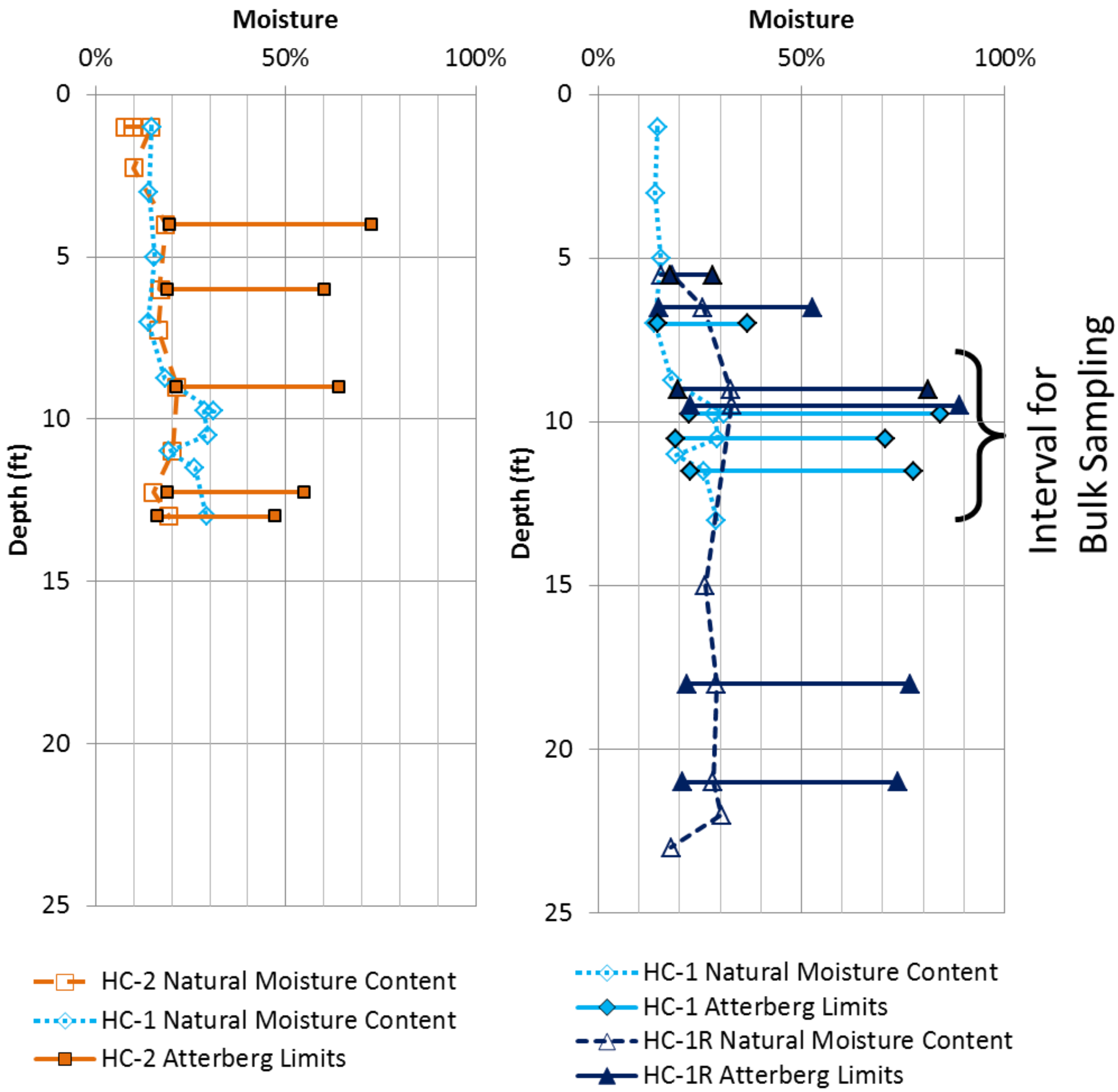


Figure 5.2.3: In-situ moisture content and Atterberg Limits for samples from borings HC-1, HC-2 and HC-1R.

Tests to define the grain size distribution were performed using a combination of wet-sieve analysis and hydrometer testing on samples collected at depths ranging from 8 to 10 ft below the ground surface. Figure 5.2.4 shows the measured grain size distribution for this material. Figure 5.2.5 shows a view of the sampling location and Figure 5.2.6 shows the bulk material collected.

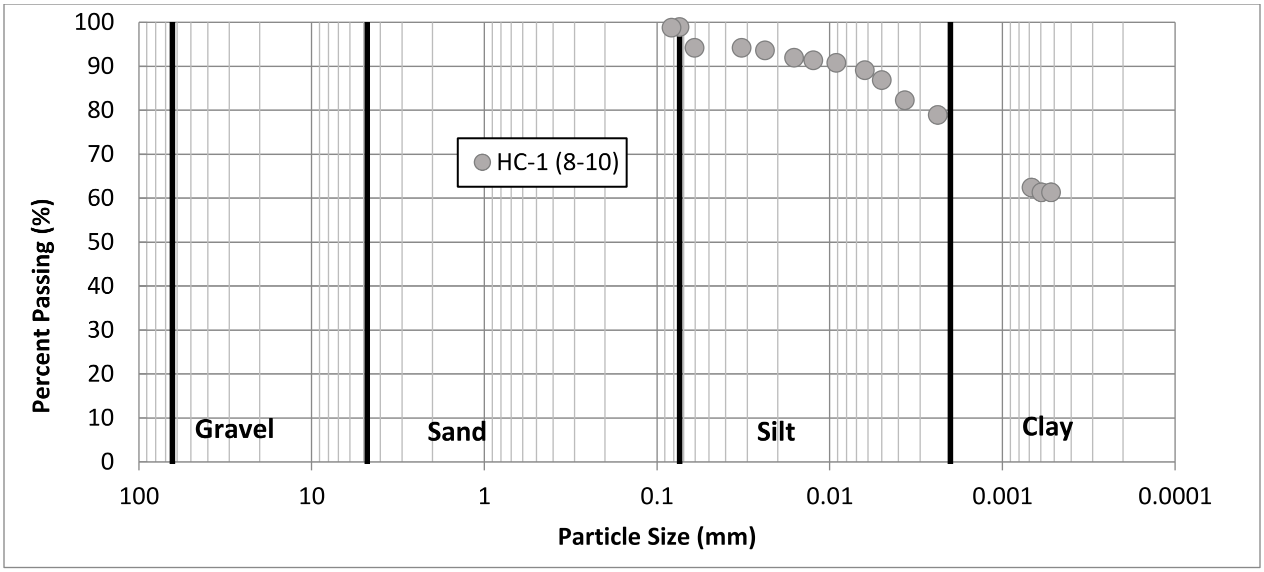


Figure 5.2.4: Grain size distribution for material recovered from I-35 and Hester's Crossing.

Figure 5.2.5: Location of sampling.

Figure 5.2.6: Bulk material collected from 10 foot depth.

5.3. FM Road 2

5.3.1. Sections 1–12 (Westbound)

Borings were collected from FM 2 during the summer of 2018 at each of 16 test sections established in Zornberg et al (2012) to evaluate the PVR using centrifuge testing on intact samples. Because the site has highly variable subsurface soil types, this also contributed to the evaluation of the effects of plasticity on swelling contained in Chapter 3. This site is situated southeast of Navasota in Grimes County, TX, as shown in Figure 5.3.1.

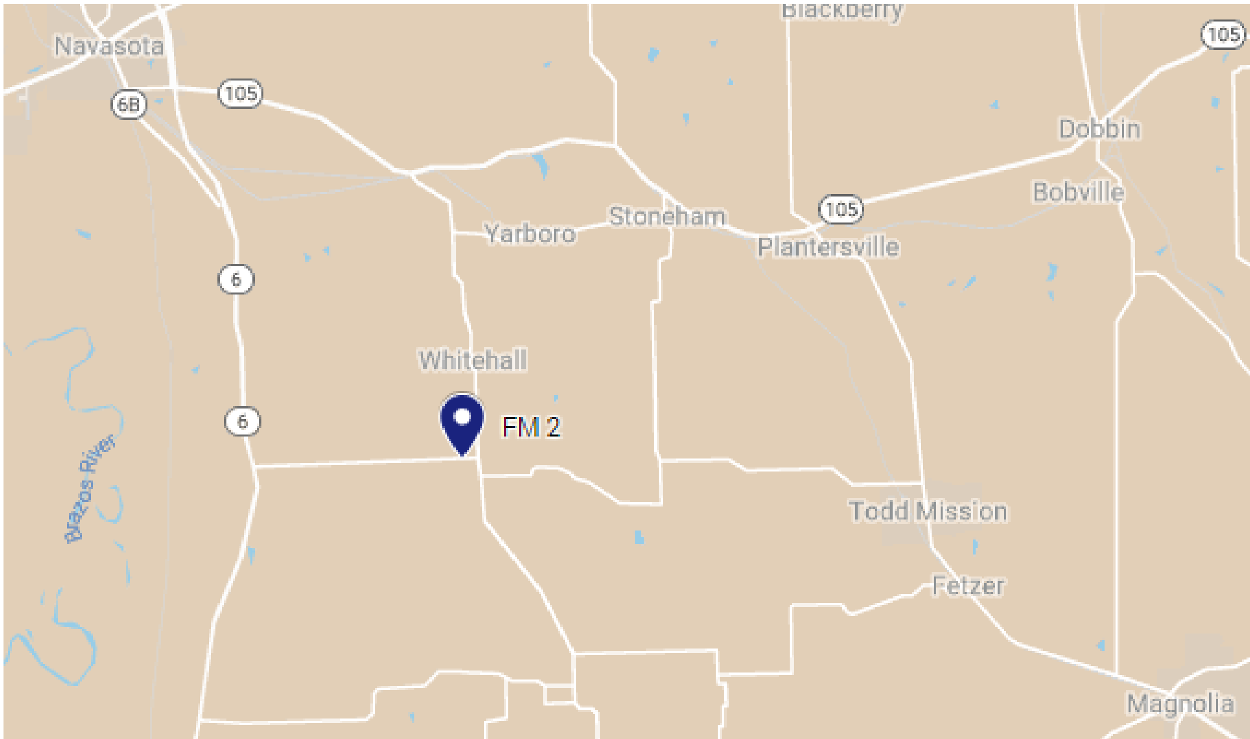
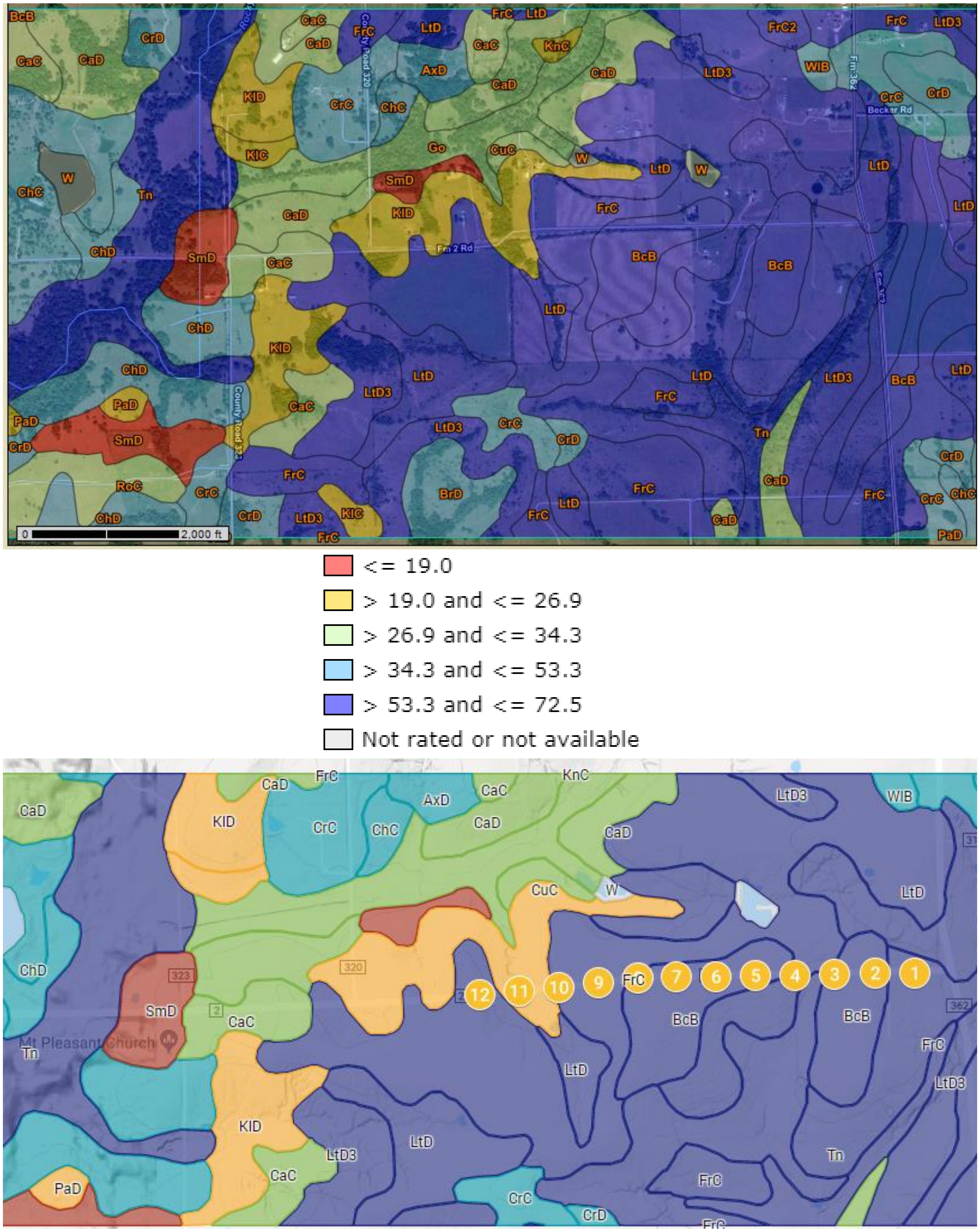


Figure 5.3.1: Site location map of FM 2 in Grimes County, TX.

In general, the site is situated on a clay upland with variable surficial clay soils intersected by numerous sand seams, and underlain by a reasonably consistent clay shale.

A generalized map of the estimated average liquid limit for the top 10 feet is shown in Figure 5.3.2, along with the boring locations for sections 1-12. The majority of the borings land in layers with relatively high plasticity, although sections 10-12 land in a region near the boundary of a significant change in soil type.



(a)

(b)

Figure 5.3.2: (a) USDA depth averaged liquid limit; and (b) showing boring locations in eastern sections of FM 2.

Table 5.3.1 lists the borings conducted from the eastern 12 test sections at the FM 2 site. Shelby tube samples were collected to a depth of 10 feet below the ground surface. The in-situ moisture content and dry density were determined using the retrieved soil samples. Texas Swell Tests were performed on undisturbed samples after being moisture-adjusted in an environmental chamber to bring each specimen's moisture content from its in-situ value to that corresponding to the initial condition for centrifuge testing as detailed in Section 2.2.3.

PVR values from the centrifuge swelling data are reported in Table 5.3.1, while the swelling data used in each evaluation is reported in Appendix B.

Table 5.3.1: Borings retrieved from FM 2 East.

Boring	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Centrifuge Testing	Soluble Sulfate Content	Lime-stabilization Characterization	PVR (Method 6048)	Site Class
1	Tn	X	X		X			2.89	High
2	BcB	X	X		X			2.68	High
3	BcB	X	X		X			4.84	High
4	FrC	X	X		X			5.24	High
5	BcB	X	X		X			4.93	High
6	BcB	X	X		X			6.15	Severe
7	BcB	X	X		X			3.21	High
8	BcB	X	X		X			5.24	High
9	FrC	X	X		X			2.82	High
10	FrC	X	X		X			2.82	High
11	KlD	X	X		X			1.34	Minimal
12	FrC	X	X		X			2.09	High

Figure 5.3.3 shows the original test section layout at FM 2. The borings here correspond to Test Sections 1–16 in the figure. Additionally, Borings 1–4 and 9–10 correspond to sections without lime treatment in the sub-base, while sections 6–8 and 13–16 involve a lime-treated subbase.

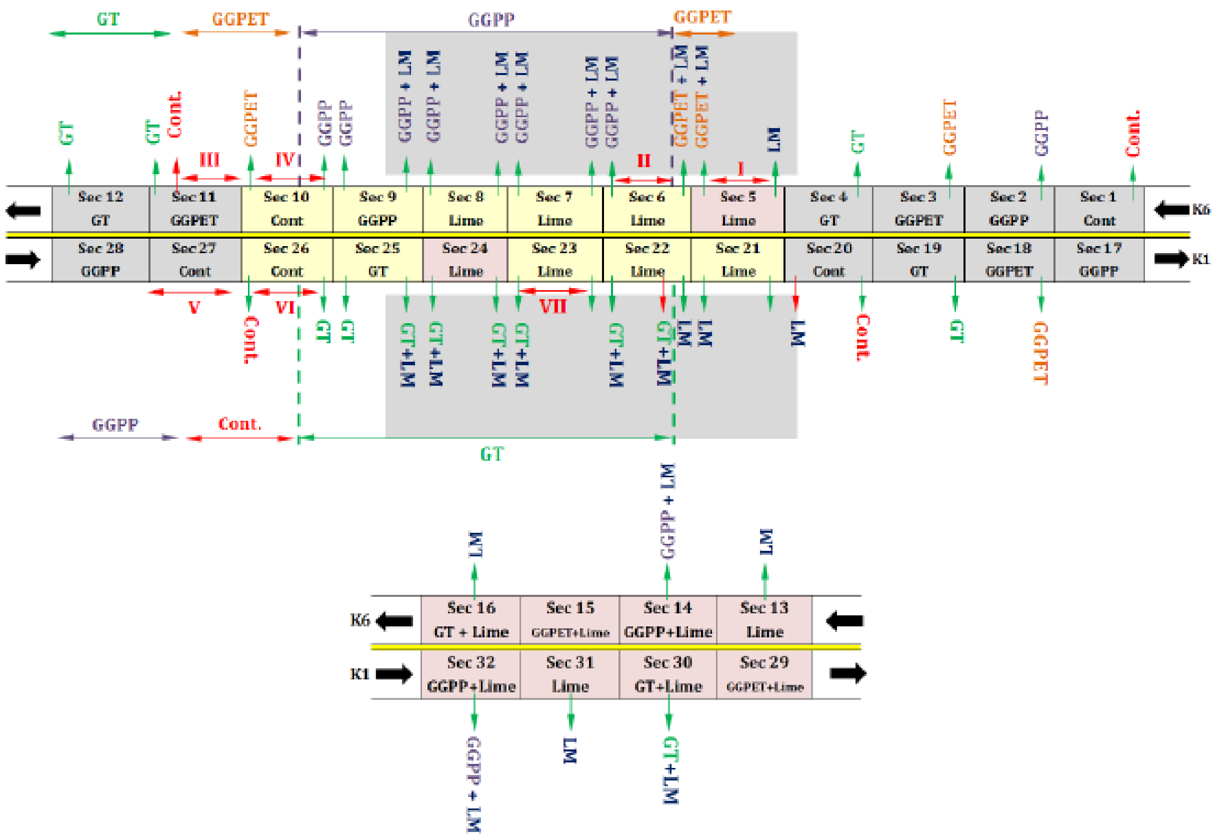


Figure 5.3.3: Test section layout at FM 2 (Zornberg et. al. 2012).

Visual description of the soil materials collected in each boring log, the Atterberg Limits (PL and LL) and the in-situ moisture content at each depth are plotted in Figure 5.3.4. Overall, the deep layers are composed of a reasonably consistent weathered shale with a high plasticity, while the upper layers are reasonably heterogeneous, often containing sand and gravel lenses intersecting the surficial clay soils. No significant difference in the subgrade soils is observed between sections with a lime-treated subbase and those without. Additionally, the PVR results predicted at the location of each boring indicate that the PVR value is primarily governed by the contribution from the comparatively deep soil stratum, without being impacted by the lime treatment, as this was used only to treat the subbase material.

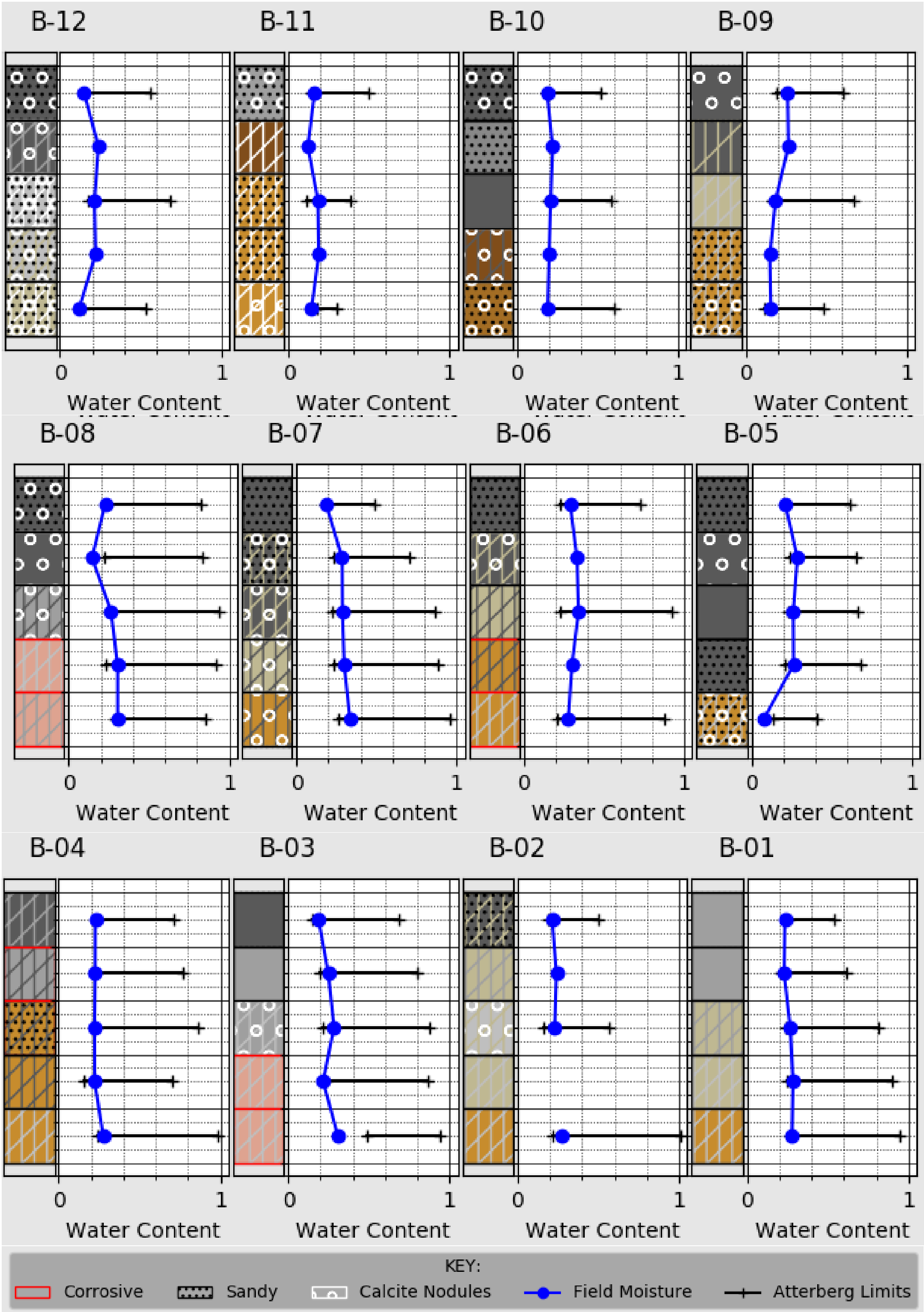
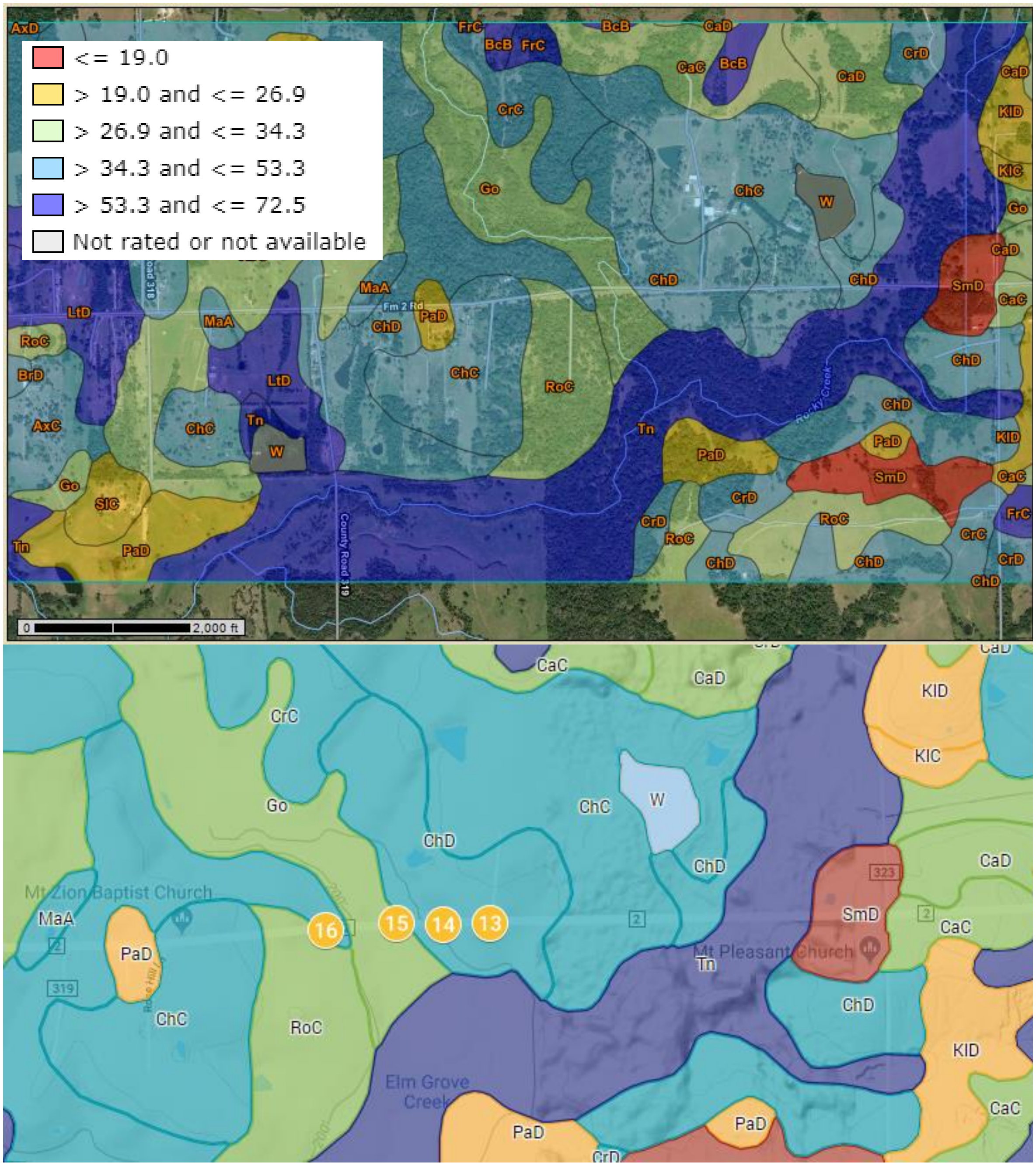


Figure 5.3.4: Boring logs from FM 2 East.

5.3.2. Sections 13–16 (Westbound)

Borings 13–16 correspond to Sections 13–16 of Zornberg et al (2012). These sections contained lime-treated sub-base and were reinforced with geogrids. Figure 5.3.5 shows the soil types encountered at these locations:



(a)

(b)

Figure 5.3.5: (a) USDA soil map at FM 2 shaded by liquid limit; and (b) showing boring locations in western sections at FM 2.

Table 5.3.2 contains a list of the borings retrieved and characterization tests performed. In general, the plasticity and swelling measured on soils in the western sections was less than in the eastern sections. These swell test results are also covered in Section 2.2.3, and the swell-stress data used in the PVR prediction is included in Appendix B. As with the eastern sections at FM 2, it should be observed that the centrifuge PVR values are driven by the subgrade soils, so that soils with higher plasticity and measured swelling values at depth lead to higher PVR values.

Table 5.3.2: Table of borings at FM 2 West.

Boring	Location	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Centrifuge Testing	Soluble Sulfate Content	Lime-stabilization Characterization	PVR (Method 6048)	Site Class
13		ChD	X	X		X			0.05	Minimal
14		ChD	X	X		X			0.05	Minimal
15		Go	X	X		X			2.46	High
16		RoC	X	X		X			3.52	High

The log information for these borings is provided in Figure 5.3.6:

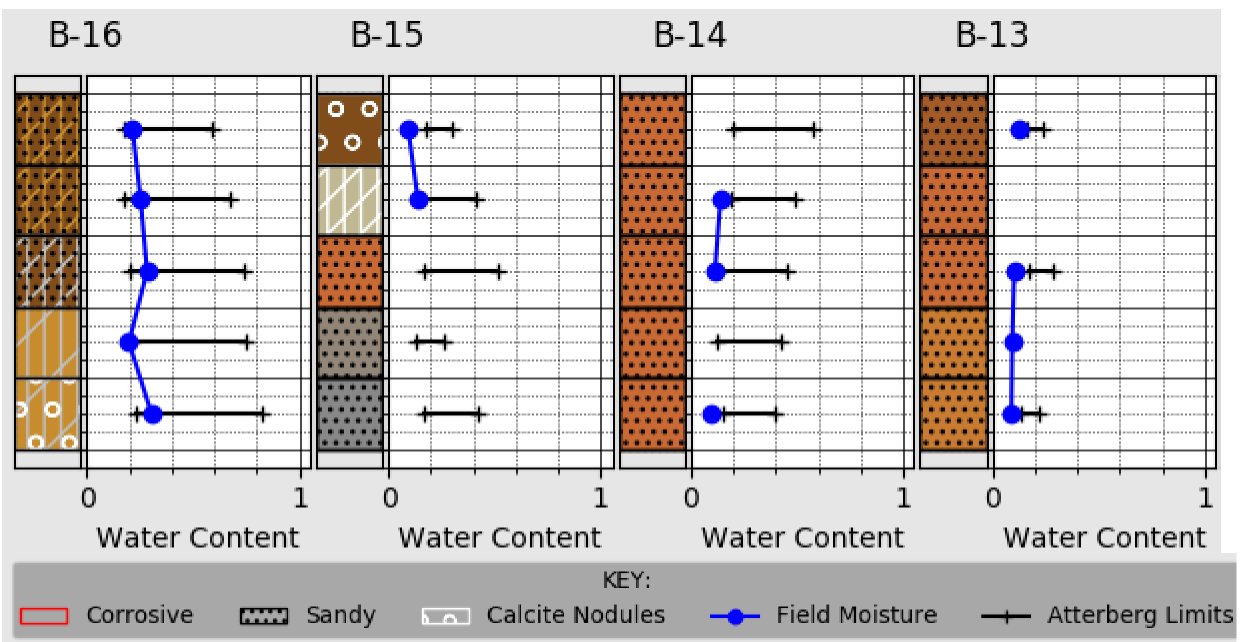


Figure 5.3.6: Boring log information from FM 2 West.

5.4. Turnersville Rd

Turnersville Road is a county road located east of the city of Buda, in Hays County, Texas. This site was surveyed by Zheng (2018) over a period of 2 years. Borings were collected to evaluate the PVR at each test section included in that monitoring program. The site location is shown in Figure 5.4.1:

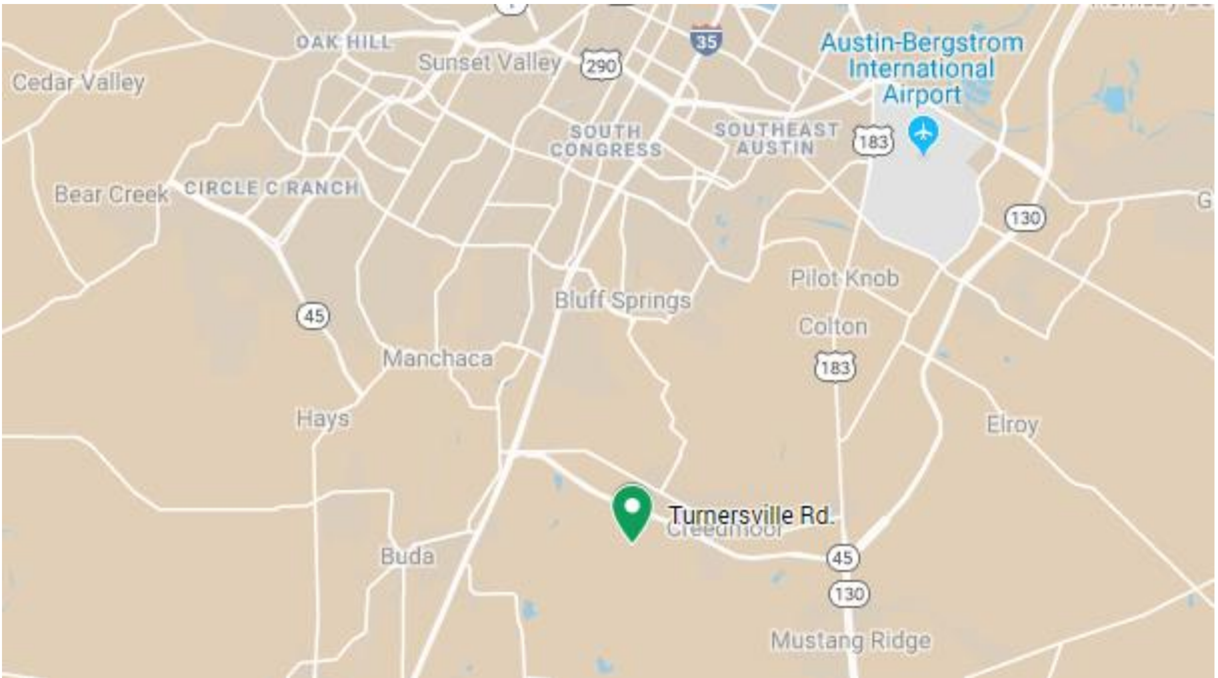
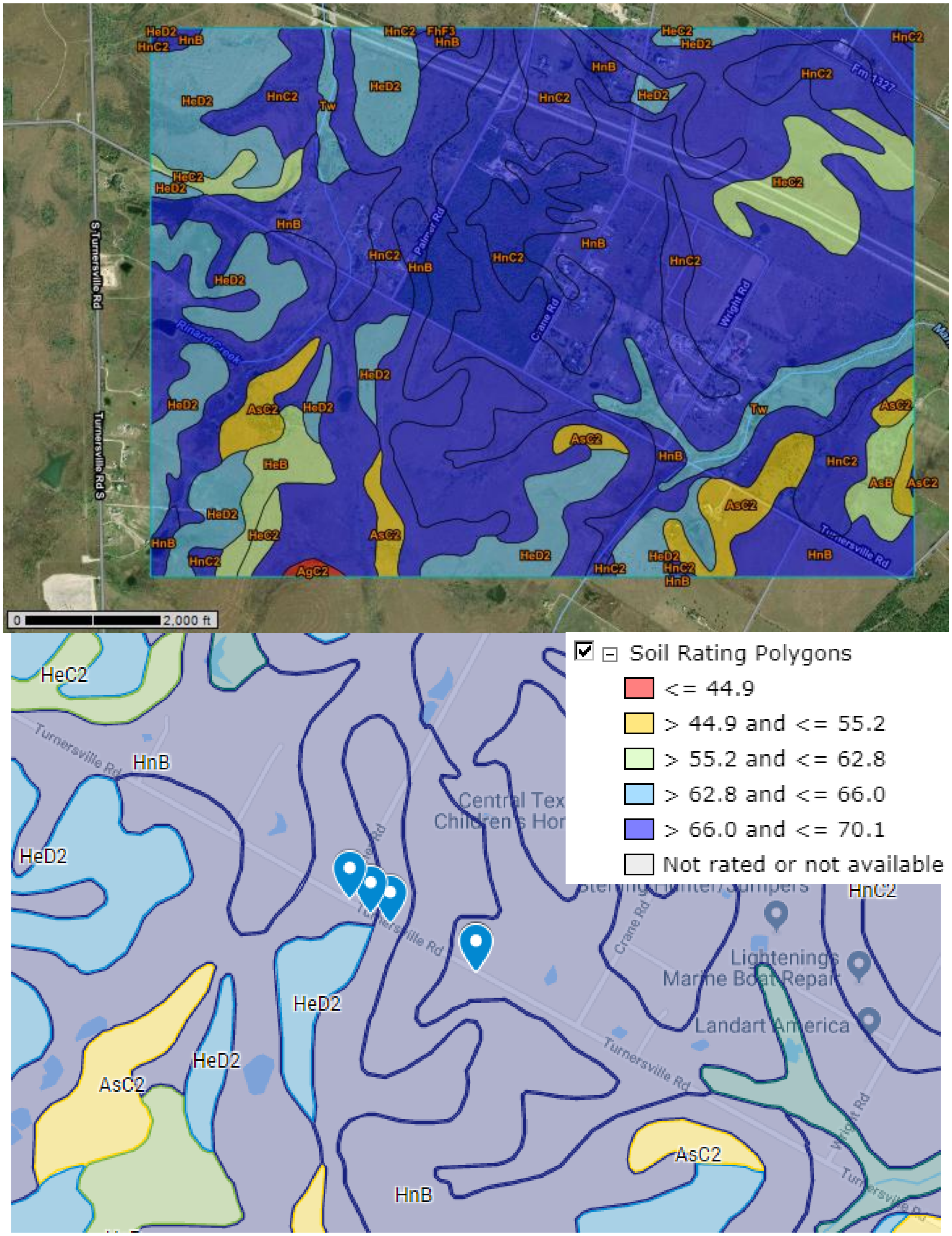


Figure 5.4.1: Site location of Turnersville Rd.

Figure 5.4.2 shows the USDA soil survey information shaded by liquid limit averaged over the top 10 feet of depth. Borings at this site landed entirely within the Houston Black clay.



(a)

(b)

Figure 5.4.2: (a) USDA soil survey overlay shaded by mapped liquid limit; and (b) boring locations.

Table 5.4.1 contains a list of the borings retrieved, and the list of soil characterization tests performed. Figure 5.4.3 shows samples from this site during processing. PVR values were calculated using centrifuge swell test results on representative materials from the group of borings. Table 5.4.2–Table 5.4.4 show the measured index values for each soil. These results indicate that there are three major soil types at this site: a dark-colored high-plasticity clay, underlain by a thin light-colored zone of low plasticity with abundant calcite, followed by a moderately high plasticity clay also with abundant calcite. The swell-stress data from which the PVR prediction has been calculated are included in Appendix B.

Table 5.4.1: Table of borings and tests conducted.

Boring	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization	PVR (Method 6048)	PVR Site Class
1	HnB	X	X		X	X			5.65	Severe
2	HnB	X	X		X	X			5.74	Severe
3	HnB	X	X			X			4.06	High
4	HnC2	X	X		X	X			2.73	High
5	HnC2	X	X			X			2.74	High

Figure 5.4.3: Processed samples from Turnersville Rd.

Table 5.4.2: Soil materials identified at Turnersville Rd.

KEY	Soil Type	Average LL	Average PI
Material 1	Black CH	72	51
Material 2	White to Tan CL	46	30
Material 3	White to Tan CH	58	41

Table 5.4.3: Liquid limit values for selected intervals.

Liquid Limit					
Boring -> / Depth	B1	B2	B3	B4	B5
0-2 FT	77	73		45	62
2-4 FT		66	44		
4-6 FT	73	48		53	
6-8 FT		57			
8-10 FT	61	57			

Table 5.4.4: Plasticity index values for selected intervals.

Plasticity Index					
Boring -> / Depth	B1	B2	B3	B4	B5
0-2 FT	55	50		28	42
2-4 FT		47	28		
4-6 FT	53	33		35	
6-8 FT		43			
8-10 FT	42	42			

Table 5.4.5: Intervals for centrifuge testing.

Centrifuge Tests					
Boring -> / Depth	B1	B2	B3	B4	B5
0-2 FT	X	X		X	
2-4 FT					
4-6 FT	X			X	
6-8 FT					
8-10 FT		X			

Figure 5.4.4 shows the Atterberg limits grouped by material type. Although the materials apparently comprise a continuum, it should be noted that the lowest plasticity material seems to occur in between the other two materials.

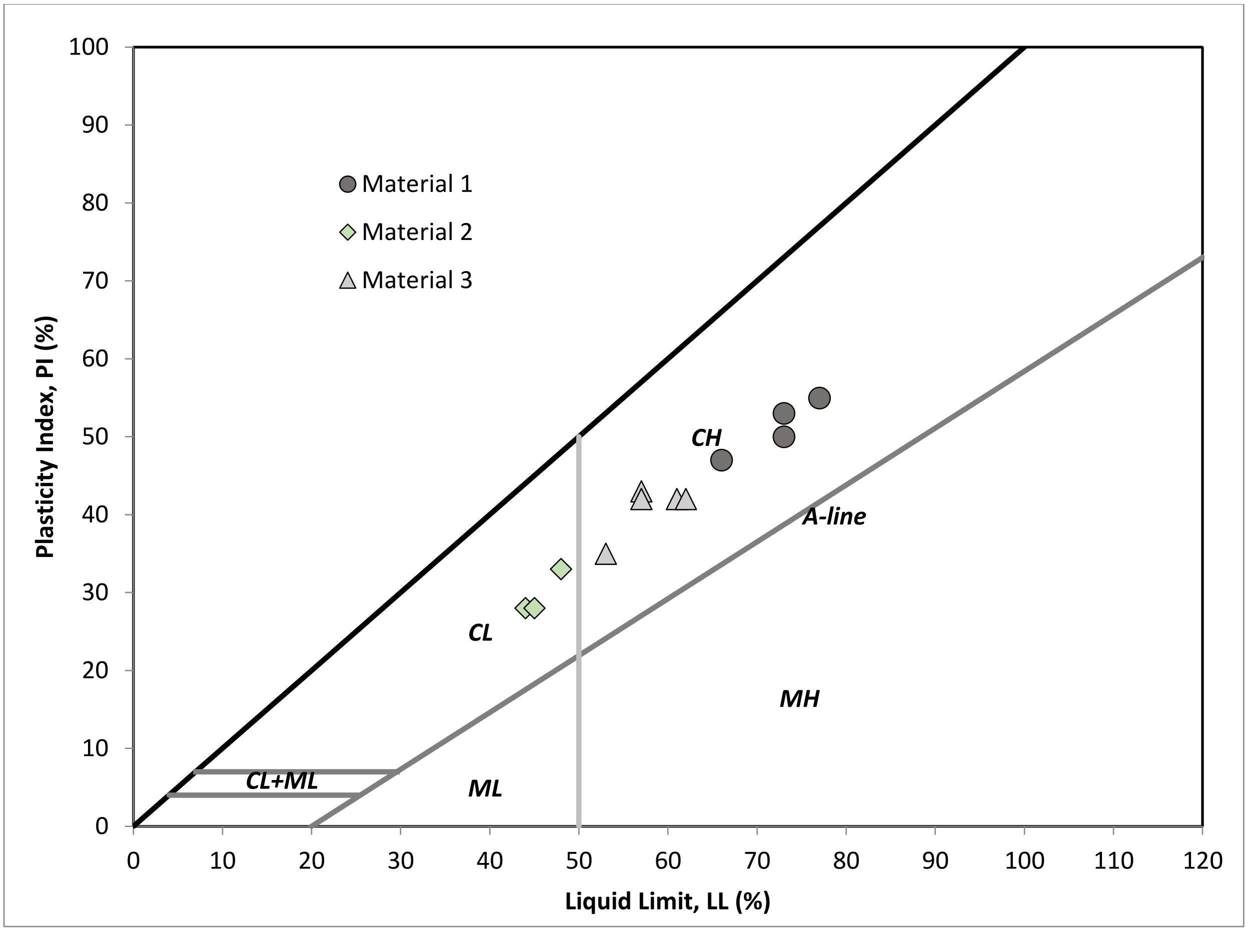


Figure 5.4.4: Plasticity data from Turnersville Rd. sections.

After initial identification of the material types in the 5 borings, depth intervals were selected to represent each material type. The initial conditions for testing were targeted as the 'dry' condition for moisture, and a dry density corresponding to a degree of saturation of 85%. The initial densities and moisture contents can be seen in Figure 5.4.6. The swelling for each soil type is shown in Figure 5.4.5. It can be seen that Material 1 (Dark CH) exhibits highly variable swelling over a very narrow range of moisture content and density, indicating that the material itself is heterogeneous, even though the liquid limit actually falls in a reasonably tight range for this material. As a consequence, the PVR was computed using actual data where specifically available, and using the average trend otherwise.

The data from Material 2 (Tan CL, represented by B4 0-2 FT) show significantly less swelling than the data from Material 1, while Material 3 (Tan CH, represented by B2 8-10 FT and B4 4-6 FT) ranges between the other two materials for swelling potential.

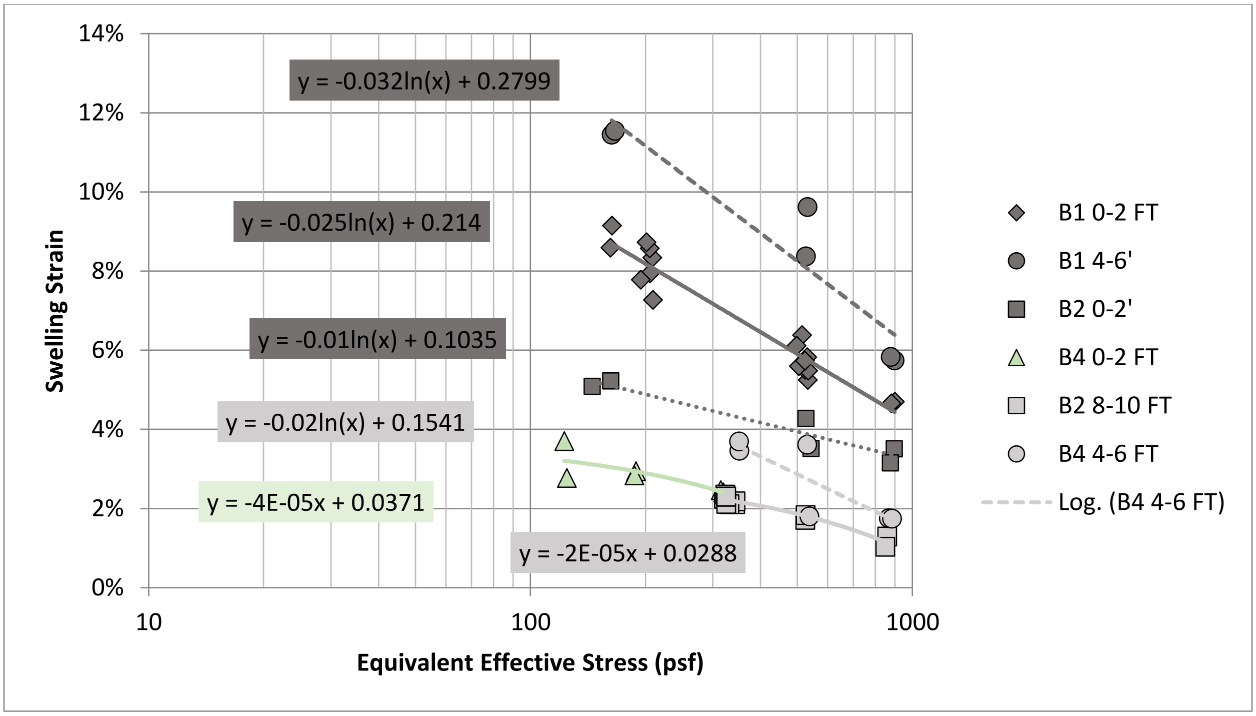


Figure 5.4.5: Swell-stress data for basic material types at Turnersville Rd.

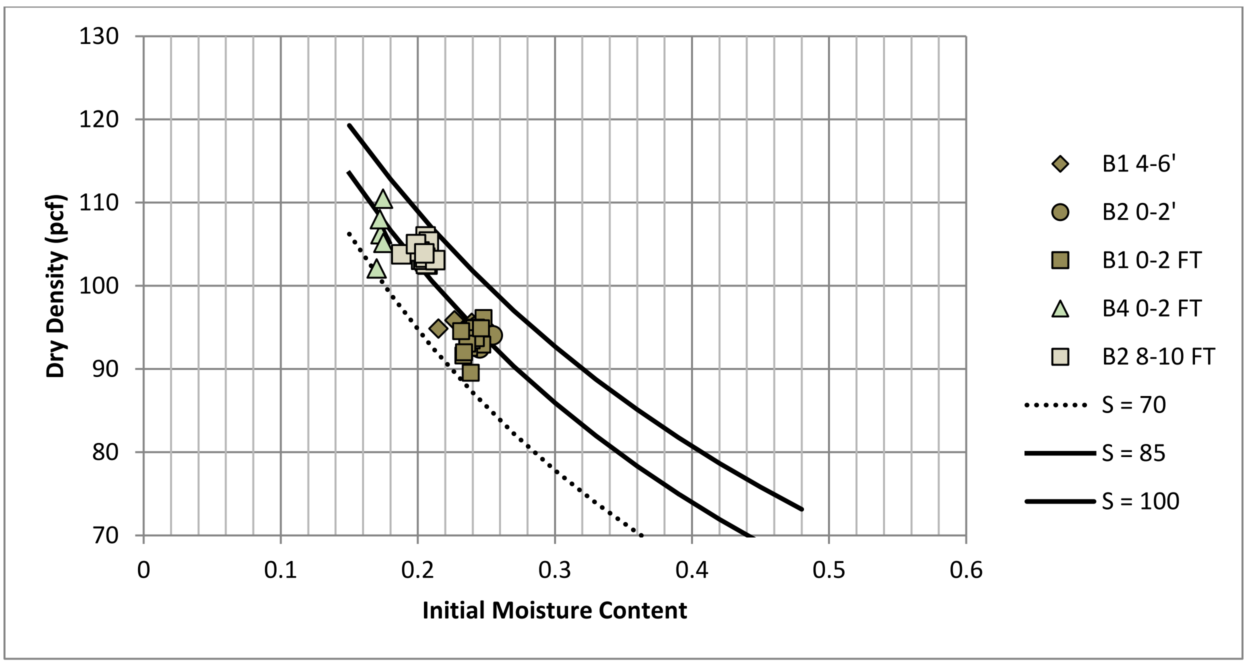


Figure 5.4.6: Initial conditions during testing for basic material types at Turnersville Rd.

5.5. FM Road 972

FM 972 is a low-volume highway located in Williamson County, Texas. The borings evaluated at FM 972 correspond to test sections monitored by Zheng (2018) east of the town of Wallburg. The surficial soils in this location contain significant gravel and coarse material, and were also used to validate the impact of the soil binder content upon the swell-shrink behavior of these soils.

Figure 5.5.1 shows the site location in relation to I-35 and State Highway 95.

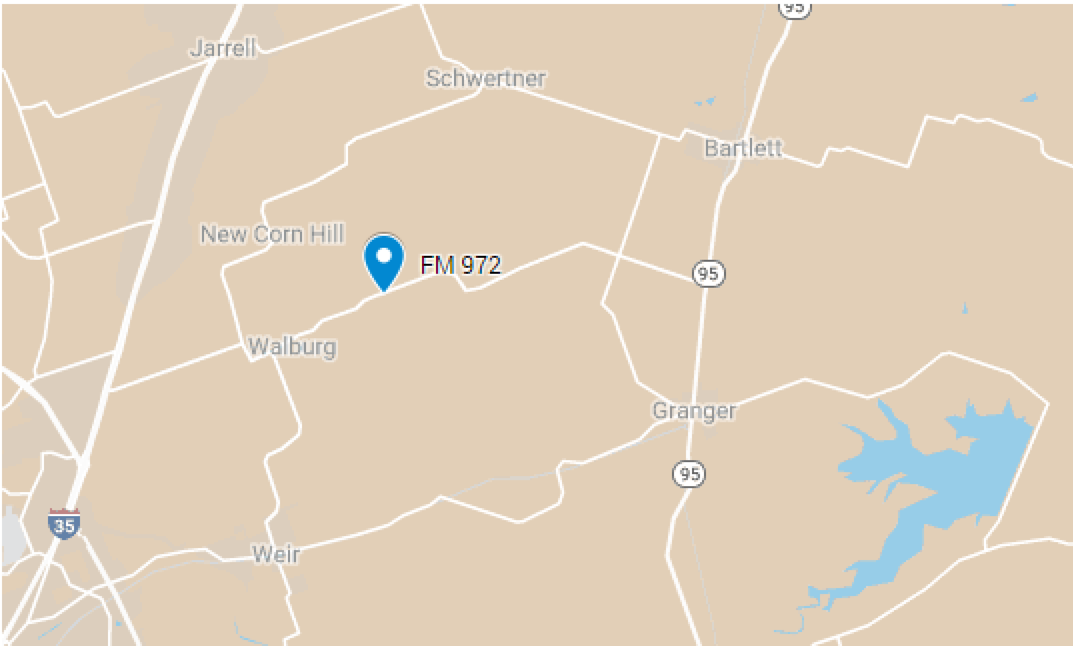
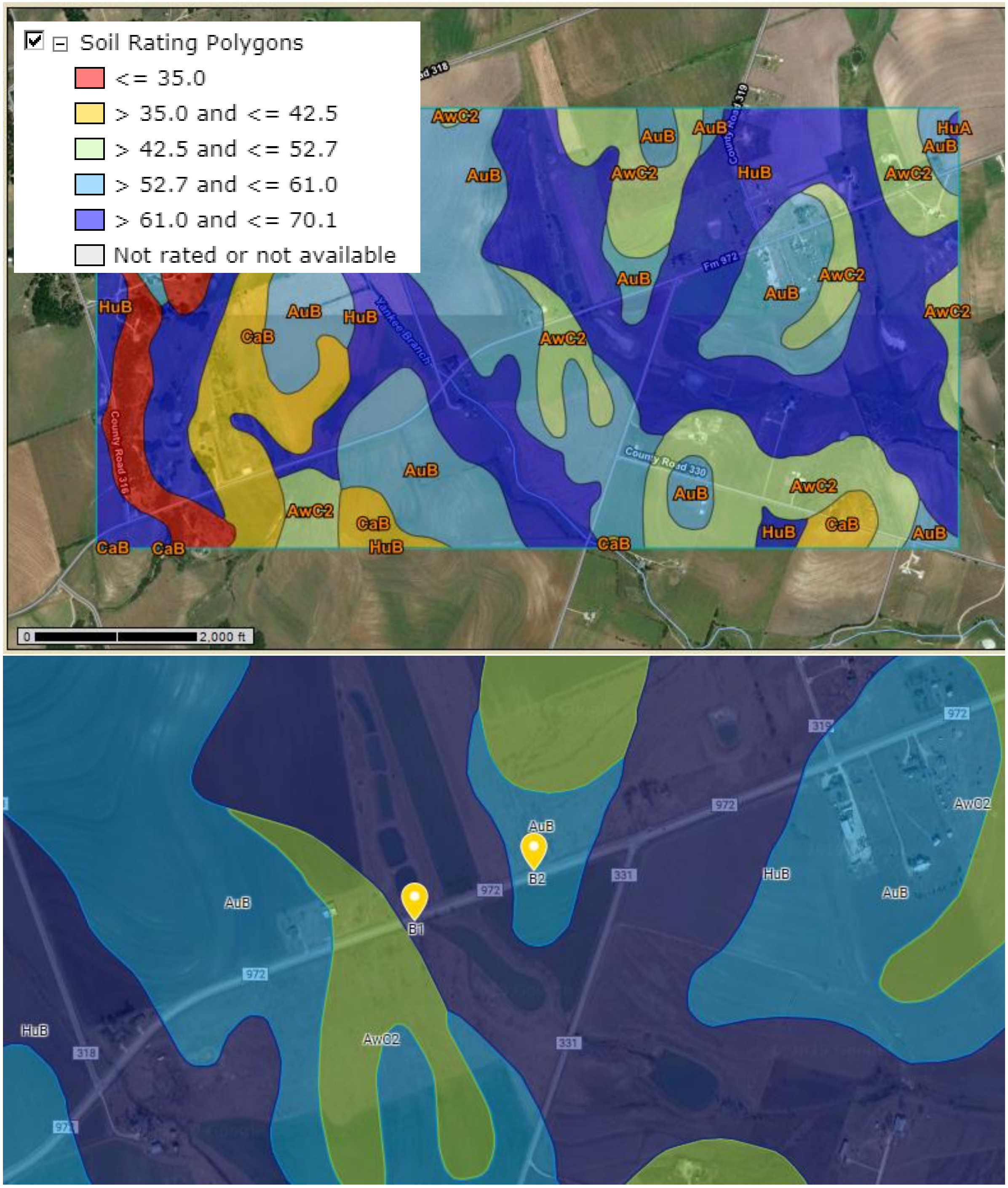


Figure 5.5.1: Location of borings at FM 972.

Figure 5.5.2 shows the USDA soil survey map of this site location shaded according to the liquid limit averages over the top 10 feet, along with the actual boring locations. Figure 5.5.3 shows the boring log information from the borings retrieved, while Table 5.5.1 shows the evaluations conducted at each boring. It can be seen that the soils in these borings contain significant amounts of calcite as well as gravel and sand. The presence of the calcium carbonate in the soil likely contributes to the lower plasticity of these soils.



(a)

(b)

Figure 5.5.2: (a) USDA soil survey map shaded by average liquid limit; and (b) boring locations superimposed on USDA soil layers at FM 972.

PVR values were calculated using the centrifuge procedure on remolded specimens from each 2-foot interval in the boring. The swell-stress data used in the calculations are included in Appendix B.

Table 5.5.1: Table of borings and tests conducted.

Boring	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization	PVR (Method 6048)	Site Class
1	AuB	X	X		X	X			2.37	High
2	HuB	X	X		X	X			2.84	High

Boring Log information is included in Figure 5.5.3. Significant soft calcite nodules were encountered in these clays, along with some sand and gravel.

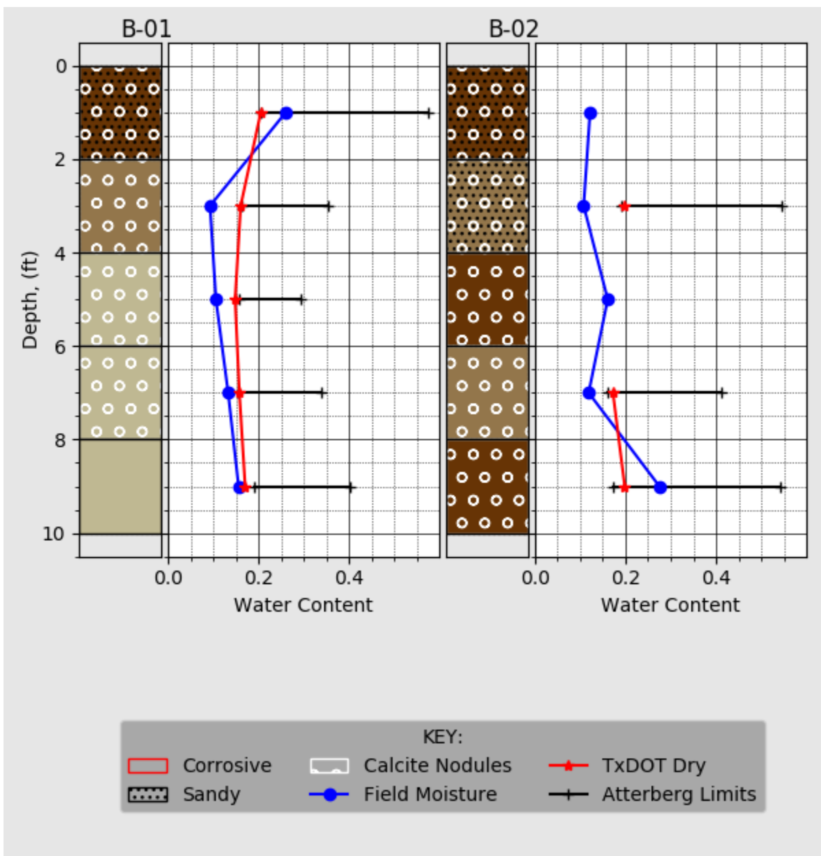


Figure 5.5.3: Boring log information for FM 972 borings 1 and 2.

Grain size distributions for the coarse fraction were performed using wet-sieve analysis. Figure 5.5.4 shows the measured grain size distributions for selected soils at this site. As shown in Section 2.2.4, the presence of the significant coarse fraction is efficient at restraining both the swelling and the shrinkage of these soils.

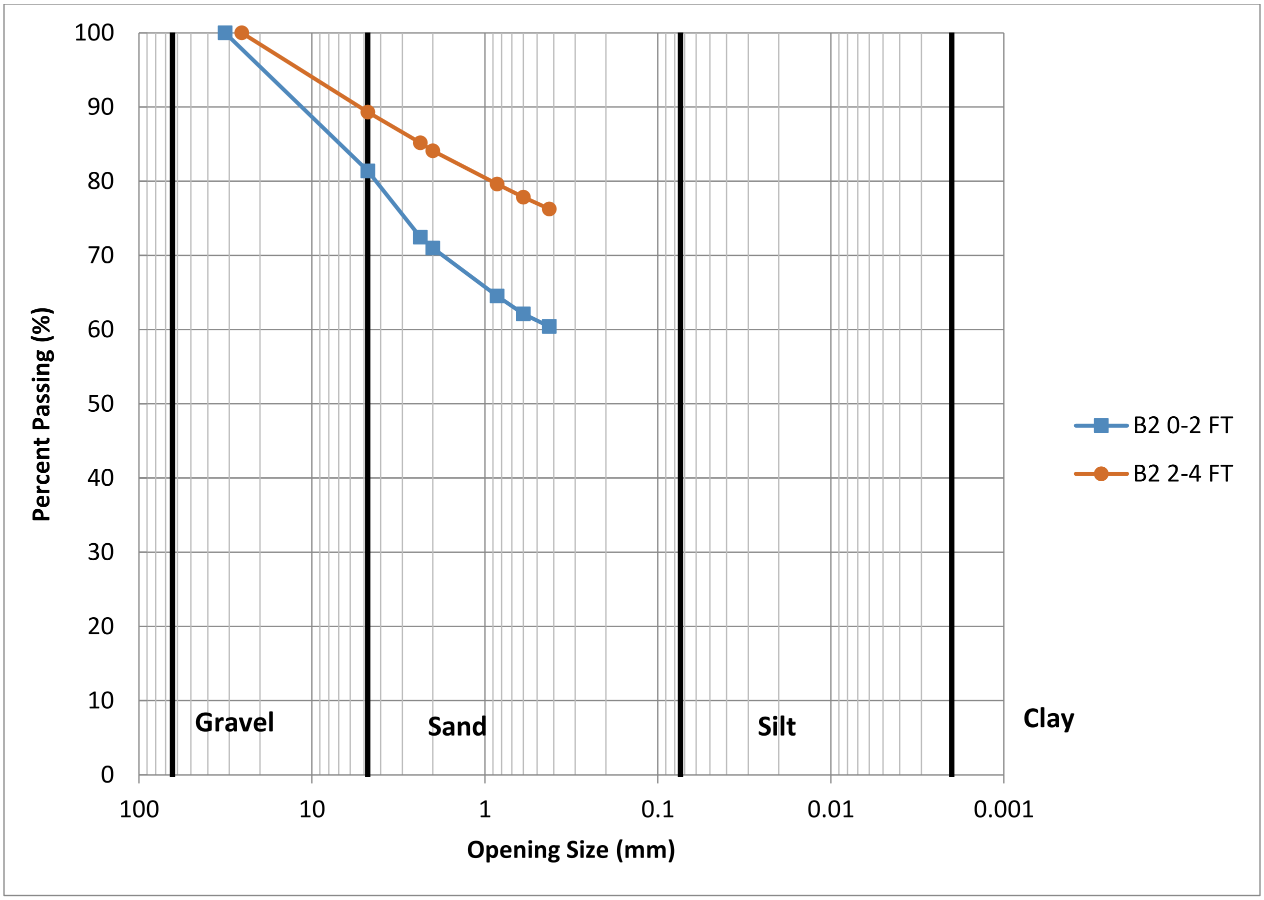


Figure 5.5.4: Grain size distributions for coarse fraction in samples from boring 2 at FM 972.

5.6. FM Road 2563 (Old Pearsall Rd) & Five Palms Drive

This site is located at the intersection of Old Pearsall Rd and Five Palms Drive in southwest San Antonio in Bexar County, Texas. This site was identified for a repair based on excessive pavement damage in the intersection. Figure 5.6.1 shows the site location in relation to Highway 410.

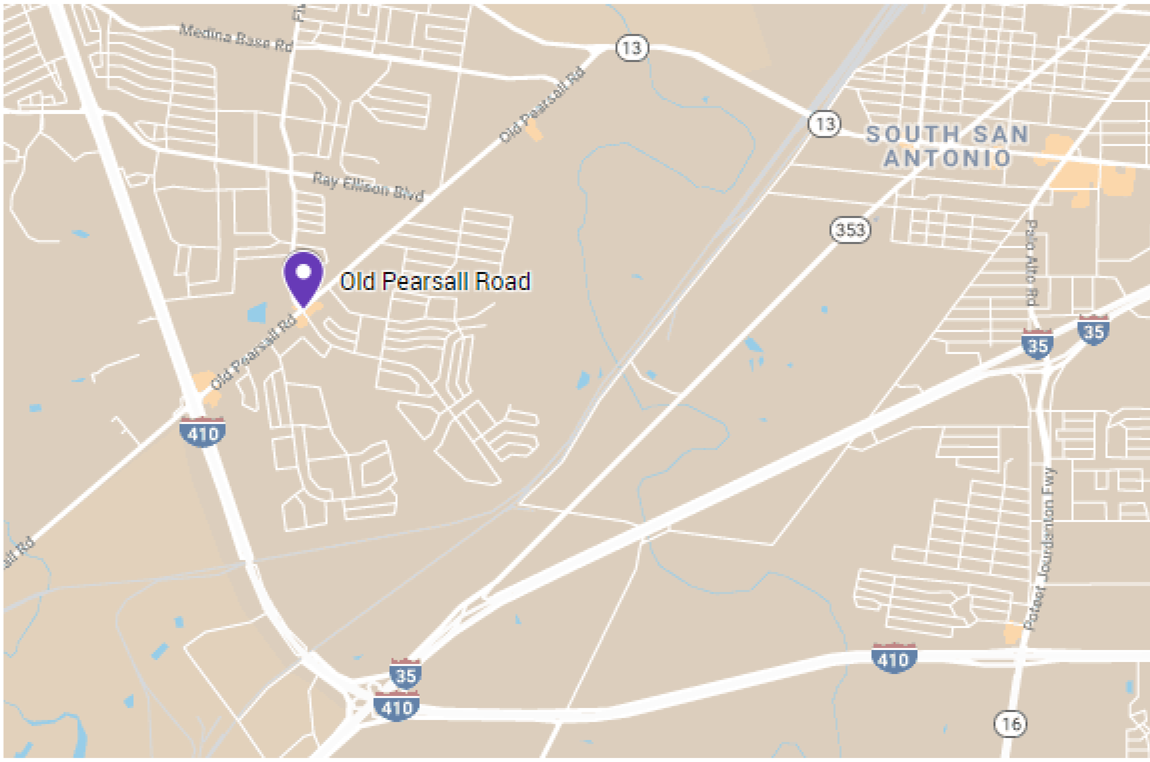
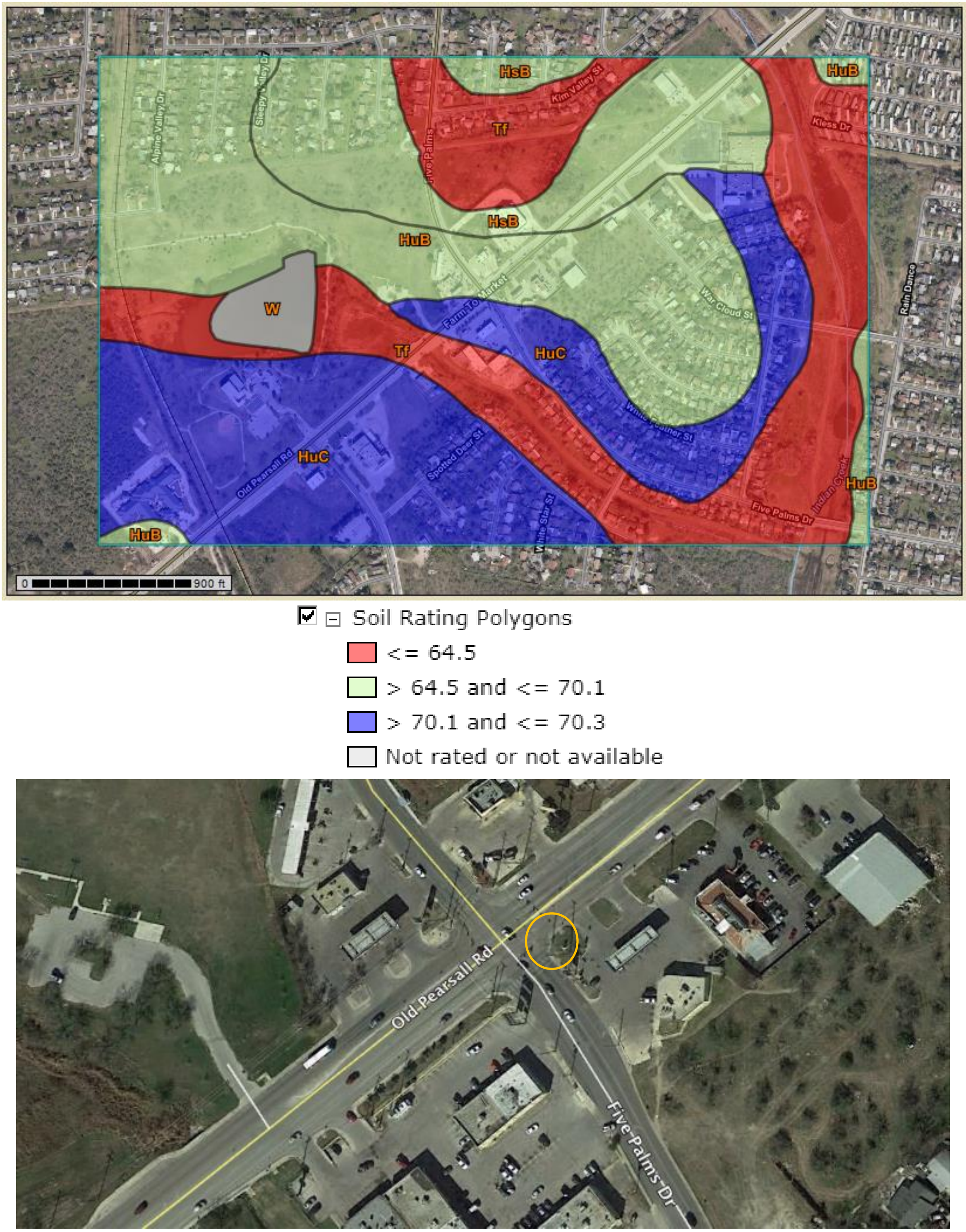


Figure 5.6.1: Site location of Pearsall Rd sampling.

Figure 5.6.2 shows the USDA soil survey map colored by the depth-averaged Liquid limit over the top 10 feet. The intersection lies within the Houston Black clay (HuC or HuB). A single boring was conducted on the eastern corner of the intersection to a depth of 10 feet. PVR predictions were made for the materials tested using Texas Swell tests. Because the sampling occurred at the corner of the intersection, there was some uncertainty concerning the actual conditions in the top three feet within the actual roadway. An alternate PVR was also calculated assuming that the top three feet within the roadway had been replaced with purely non-expansive fill, although the values measured within the top three feet of material retrieved from the site may actually be representative of the non-expansive fill.



(a)

(b)

Figure 5.6.2: Sampling location at Old Pearsall Rd.: (a) USDA soil map shaded by liquid limit; and (b) boring location.

Table 5.6.1 shows a summary of the tests conducted on samples from this boring. Additionally, Table 5.6.2 shows the results for index characterizations performed on each depth interval. The top three feet even in the corner of the intersection are composed of high plasticity clays, while the deeper strata are composed of very high plasticity clays. Figure 5.6.3 shows the measured swell-stress curves for a series of tests conducted on the material taken from this site, while Figure 5.6.4 shows the initial conditions of testing on these specimens. This data shows that the potential for swelling increases with depth for these materials, with a somewhat abrupt change occurring in the 3-4 ft interval. The swelling data on the material from the top 3 feet show that minimal swelling is to be expected from these materials (with a swelling from 1 to 3% at 100 psf), despite their high plasticity and low water content. The materials from deeper strata, however, exhibit about 10% swelling upon wetting at a stress of 100 psf; these soils are expected to contribute to the pavement damage if cyclic moisture changes can indeed occur to depths of 10 feet at this location.

Table 5.6.1: Table of borings and tests conducted.

Boring	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization	PVR (Method 6048)	Site Class	Alternate PVR (3 ft replacement)	Site Class
1	HuB	X	X	X	X	X		X	2.93	High	2.42	High

Table 5.6.2: Results of characterizations performed by depth.

Depth	Sampling Type	Characterizations Performed			
		In-situ Water Content	LL	PI	Centrifuge PVR on Remolded specimens
0’ – 1’	Thick-Walled Tube	19.1%	60	23	X
1’ – 3’	Curls	28.1%	58	22	X
3’ – 4’	Thick-Walled Tube	21.9%	98	28	X
3.5 – 4’	SPT	23.5%	96	30	X
4’ – 5’	SPT	22.3%	89	30	X
5’ – 6’	Thick-Walled Tube	25.6%	100	29	X
6’ – 7.5’	SPT	26.7%	91	30	X

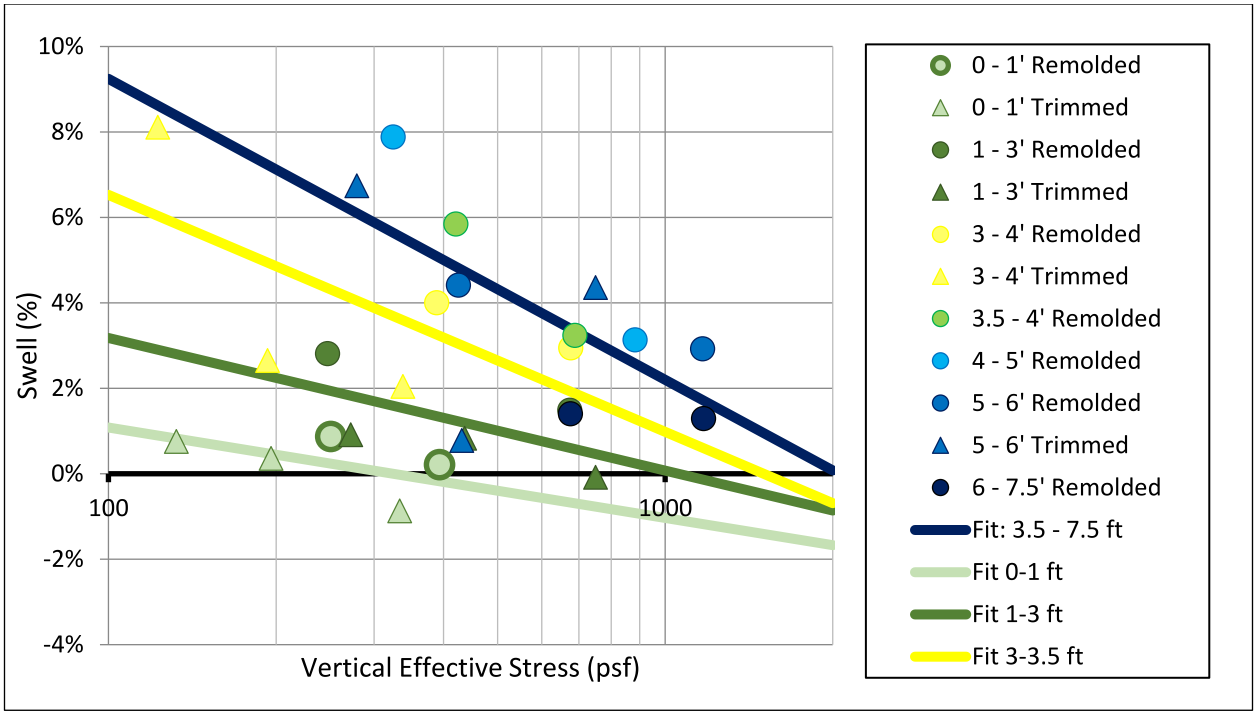


Figure 5.6.3: Swell stress data for Old Pearsall Rd.

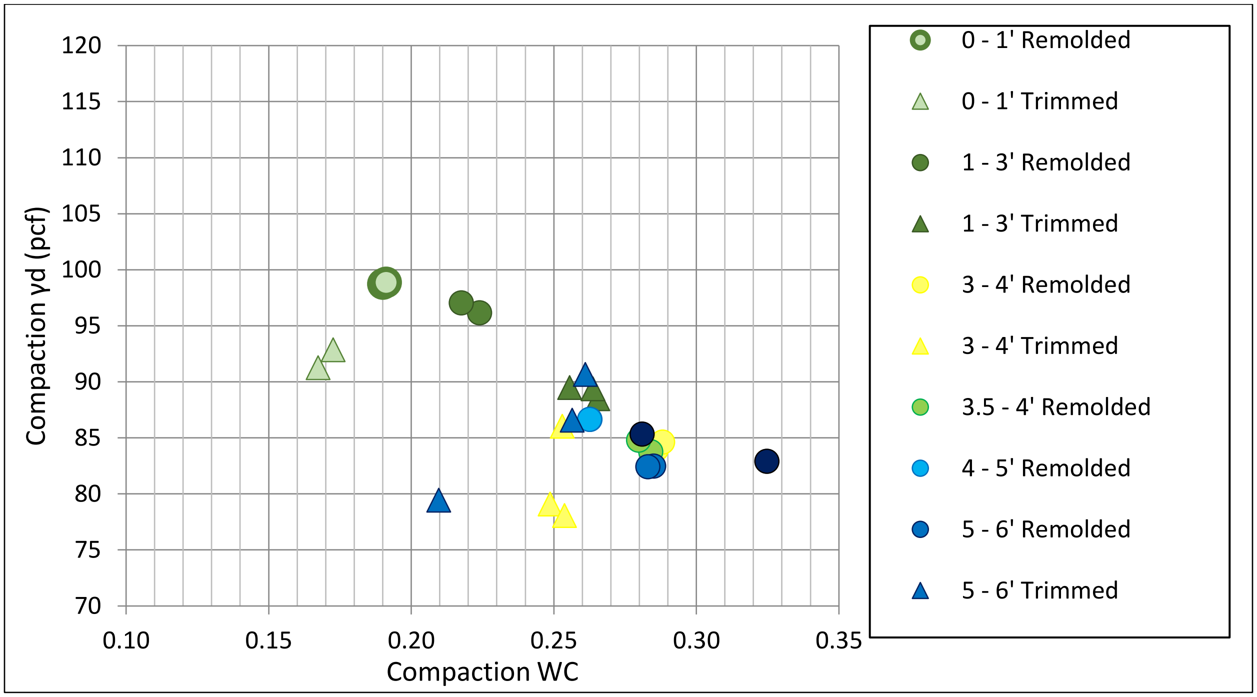


Figure 5.6.4: Initial testing conditions for specimens from Old Pearsall Rd.

Figure 5.6.5: Intersection at Old Pearsall Rd. and Five Palms Dr.

5.7. US 87 from I-10 to Rigsby Ave.

The site at US 87 (from I-10 to Rigsby Ave.) involved an existing pavement section that exhibited signs of severe deterioration. The site had been identified for reconstruction, and borings were collected in July 2017 to evaluate the PVR using Texas Swell Tests. Figure 5.7.1 shows the site location in eastern San Antonio.

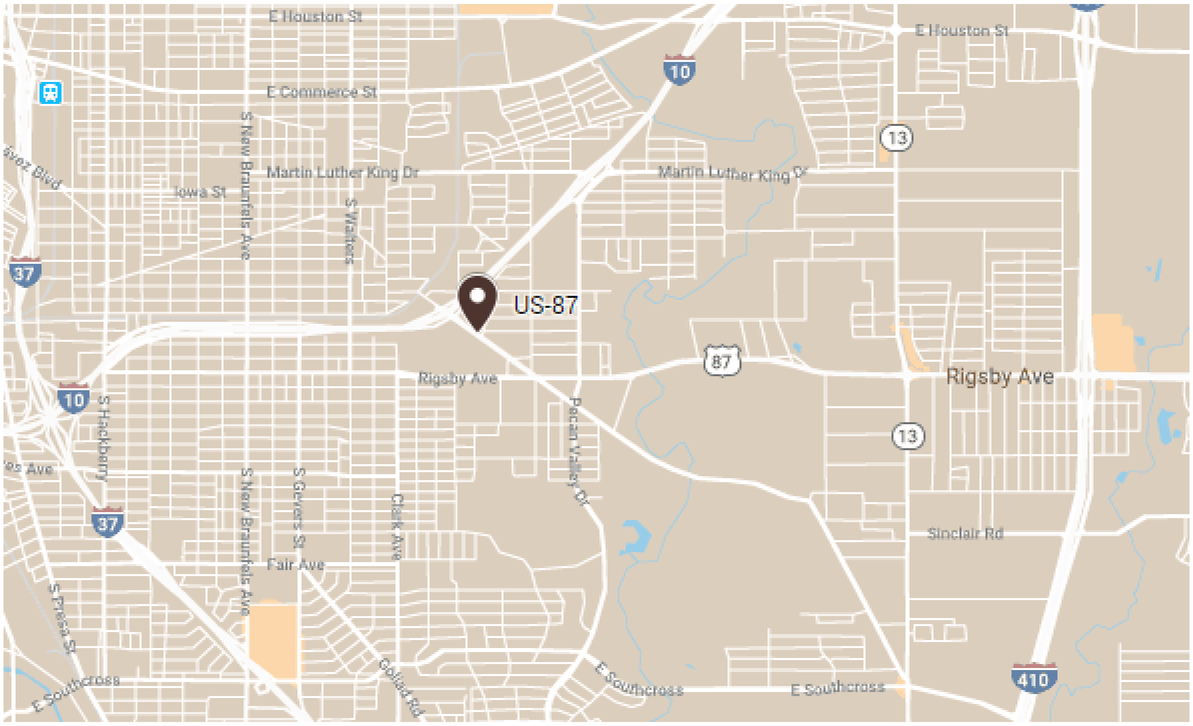
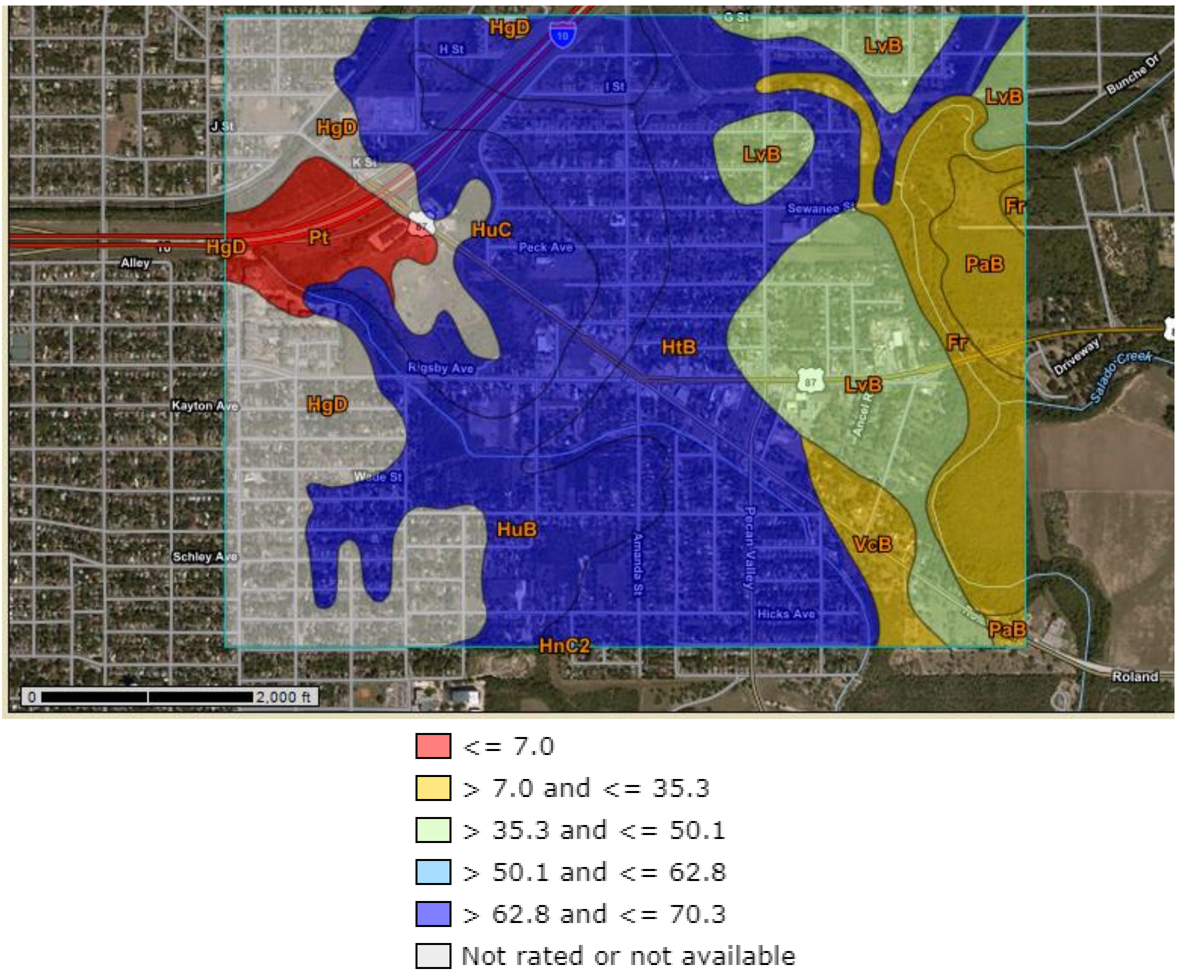
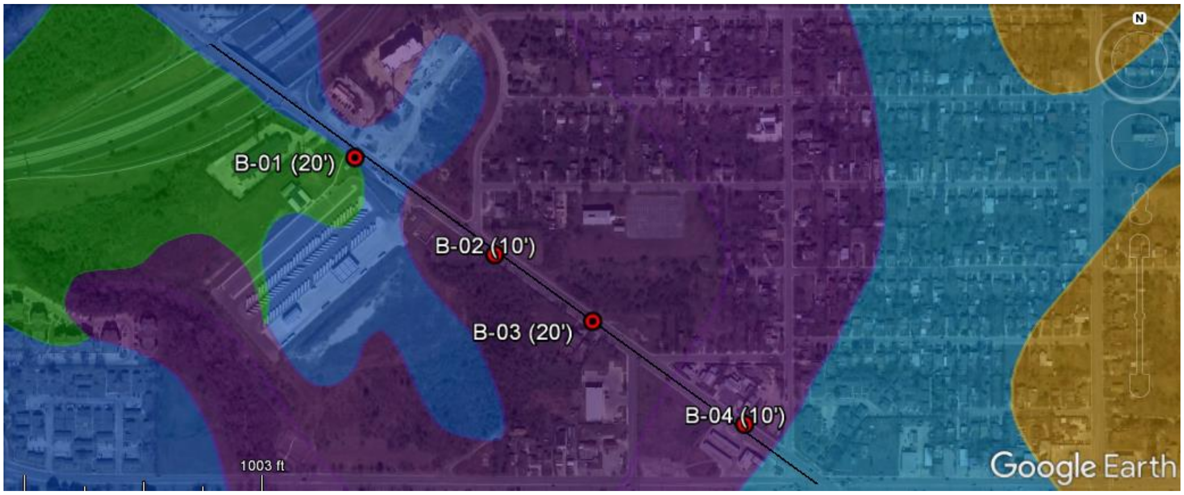


Figure 5.7.1: Site location of US 87.

The roadway at this site is on a slope, at the crest of which is a significant deposit of low plasticity clay and gravel of the Olmos Complex. Further down the slope are significant higher plasticity clay deposits mapped as Houston Black clay. Borings were spaced evenly along the project alignment, and extended to 10 or 20 feet depth as necessary. Samples were collected from Borings 1, 2, and 4 for centrifuge testing and PVR analysis. Figure 5.7.2 shows the USDA soil map shaded by the average liquid limit over the top 10 feet, as well as a Google Earth image of the site showing the boring locations, overlaid by the USDA soil map.



(a)



(b)

Figure 5.7.2: Boring locations at US 87: (a) USDA soil survey map; and (b) boring locations.

Table 5.7.1 and Table 5.7.2 contain a list of the borings retrieved and characterization tests performed. Boring 3 was not studied in detail due to improper separation of the materials retrieved from the borehole during the drilling process.

Table 5.7.1: Table of borings at US 87.

Boring	Probable USDA Soil Type	Moisture Content	Atterberg Limits	Grain Size	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization	PVR Method 6048	Site Class
B1	HgD	X	X	X	X	X			0.08	Minimal
B2	HuC	X	X		X	X		X	5.35	Severe
B3	HuC									
B4	HuB	X	X		X	X		X	5.17	Severe

Table 5.7.2: Characterizations performed at US 87.

Boring	Depth	Characterizations Performed						
		In-situ Water Content	Plastic and Liquid Limit		Sulfate Content [ppm]	Grain Size (Wet sieve or Hydrometer)	Native PVR (Centrifuge)	Lime-Treated PVR (Centrifuge)
B-01	0.5’ – 4’	5.7%						
B-01	4’ – 6’	9.5%						
B-01	6’ – 8’	10.1%	23	14			X	
B-01	13’	15.6%			764			
B-01	16’	27.4%						
B-01	18’ – 20’	26.2%	80	24			X	
B-02	0.5’ – 4’	28.7%	81	26	53	X	X	X
B-02	4’ – 6’	30.0%	89	22	231	X	X	X
B-02	6’ – 10’	30.7%	89	25		X	X	X
B-04	0.5’ – 2’	26.7%	72	21	0		X	X
B-04	2’ – 4’	28.0%	74	24			X	X
B-04	4’ – 6’	26.3%	74	22			X	X

Soil information collected from borings is included in Figure 5.7.3.

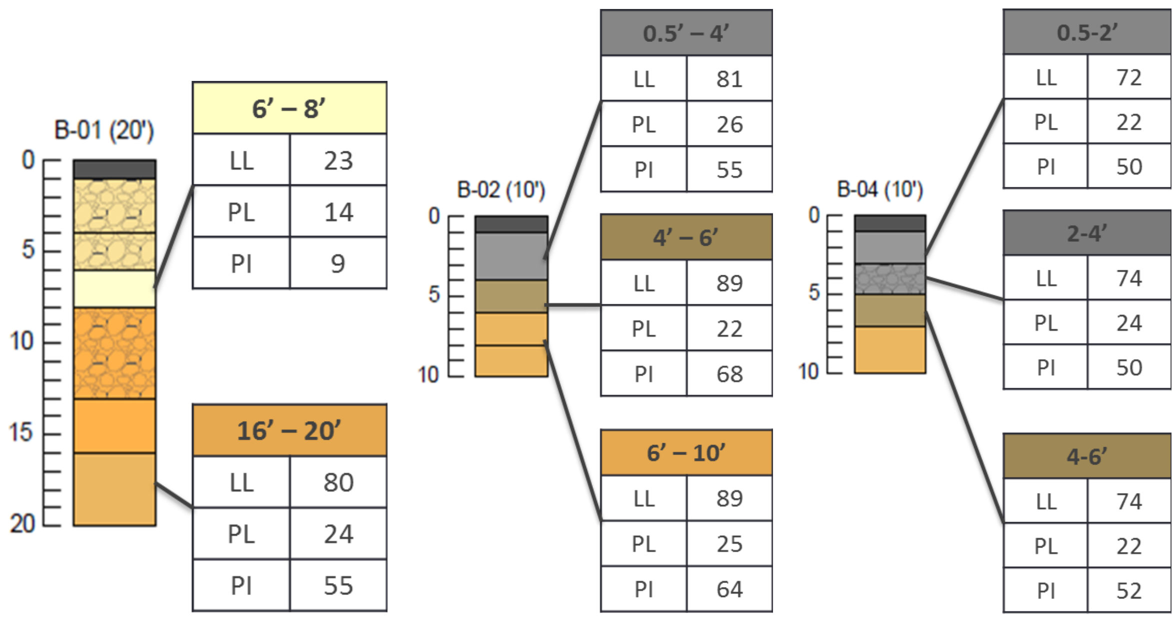


Figure 5.7.3: Identified soil layers in borings from US 87.

Grain size distributions were performed using a combination of wet-sieve analysis and hydrometer testing. Figure 5.7.4 shows the measured grain size distributions for selected soils at this site. The material on the site generally tends to grade into a finer material deeper into the slope.

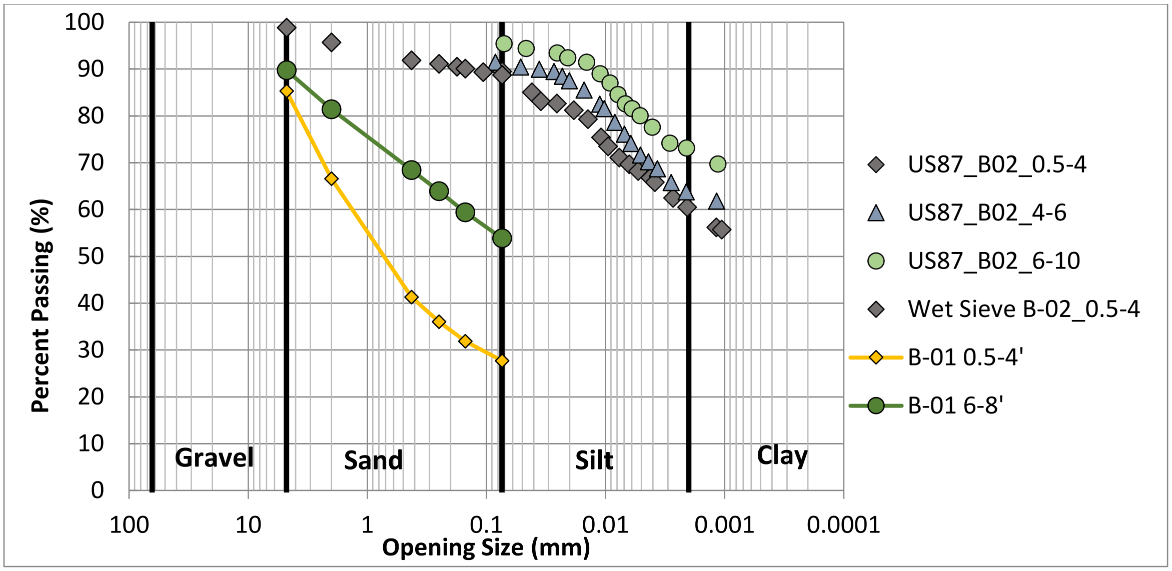


Figure 5.7.4: Grain size distributions on soils taken from borings 1 & 2 at US 87.

The soil layers inferred from the borings and from the underlying geology at the site are shown in Figure 5.7.5. It is expected that the subgrade at the crest of the slope has significant overburden from the non-plastic soils, and consequently the PVR will be low at this location.

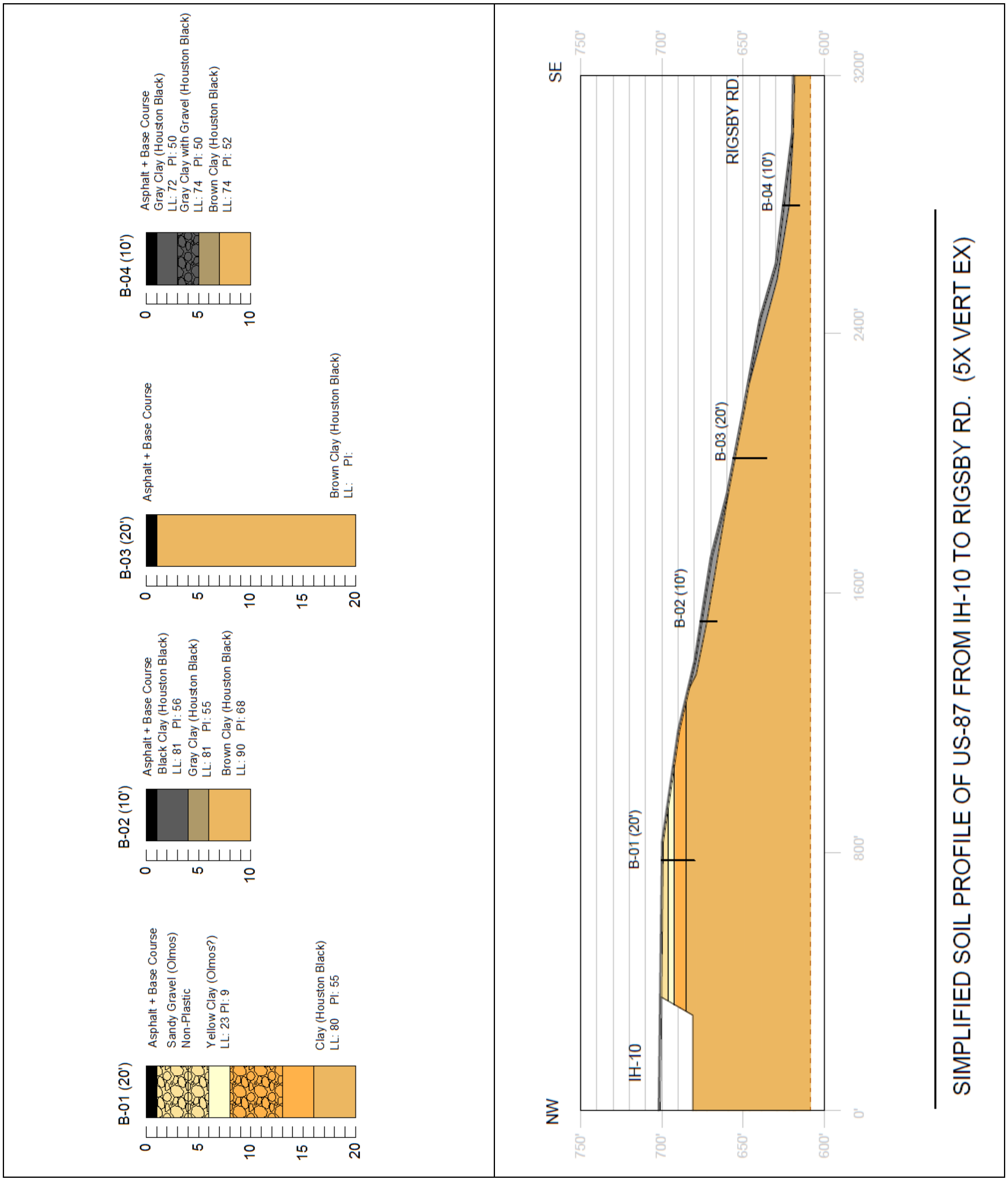


Figure 5.7.5: Approximate site cross section for US 87.

Figure 5.7.6 shows the distress observed in the vicinity of Boring 2. Extensive cracks were observed running parallel to the roadway, and extending down the slope. In addition, some of the distress features were arcuate in shape, as shown on the left-hand panel in the figure.


(a)

(b)

Figure 5.7.6: Distress in northbound lane of site: (a) facing southeast; and (b) facing northwest.

The PVR was evaluated using Texas Swell Tests on materials retrieved from the borings. In addition to evaluating the PVR corresponding to the profile of native soils, assessments were also conducted to evaluate lime treatment options. These evaluations were conducted after generating data on lime-treated specimens under low confining stresses and predicting the swell-stress curves.

Figure 5.7.7 and Figure 5.7.8 show the PVR reduction curves calculated at Borings 2 and 4 for increasing depths of treatment with lime, based on centrifuge swelling tests conducted on natural and lime-treated samples from each boring. This analysis was performed by first calculating the swell stress curves for each 2-foot layer in the boring using test data from natural soils collected from that boring. Then the top layer in the PVR calculation was substituted with the swell-stress curve generated from lime-treated swelling data. As the thickness of this treated layer is extended downward, the PVR will be reduced in proportion to the amount of material replaced with lime-treated material, except that the lime-treated material itself will contribute a small amount of expansion to the overall PVR. This calculation was repeated for each lime dosage tested in the centrifuge, as well as for an idealized non-expansive fill.

Based on this analysis, the depth of treatment required to reduce the PVR below 1 inch at Boring 2 ranges from 3 to 5 feet depending on the alternative considered, as shown in Figure 5.7.7:

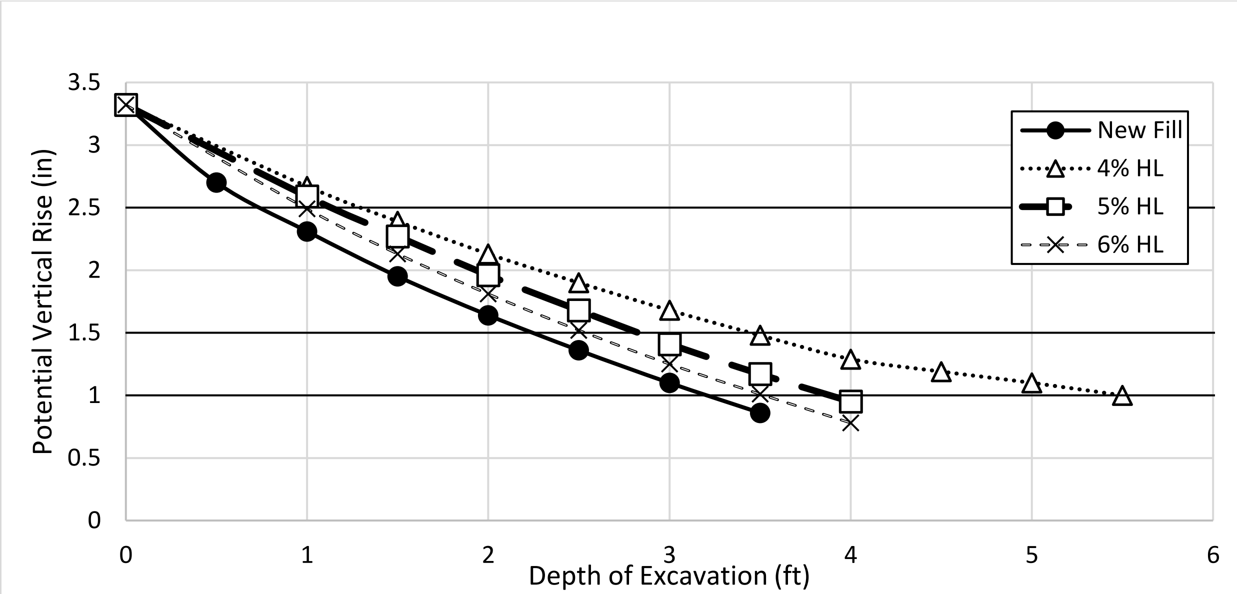


Figure 5.7.7: Treatment depth alternatives for boring 2 at US 87.

At Boring 4, the subgrade is significantly more expansive, so the depth of treatment calculated to reduce the PVR to 1.5 inch is in the range of 5 to 8 feet, as shown in Figure 5.7.8.

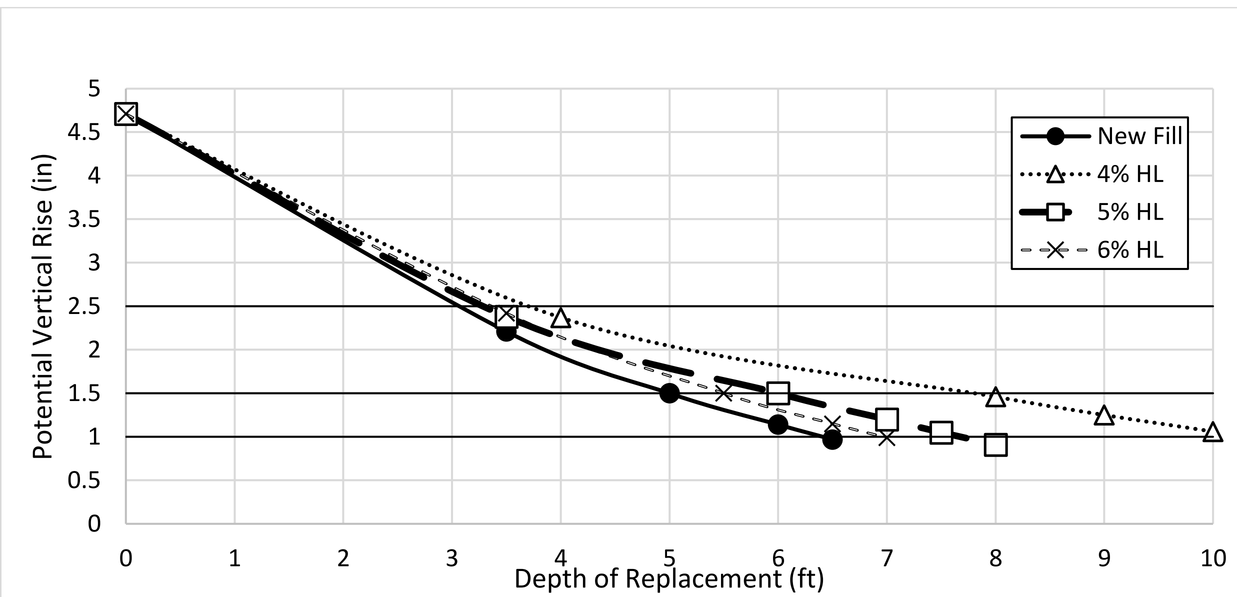


Figure 5.7.8: Treatment depth alternatives for boring 4 at US 87.

5.8. US 183 & Martin Luther King Jr. Blvd

US 183 at Martin Luther King Jr. Blvd was identified for a PVR prediction during a major rebuild of the highway, due to the lower plasticity encountered in the subgrade than anticipated. Figure 5.8.1 shows the site location east of downtown Austin. Materials from this site were used to predict the PVR using Texas Swell tests, and also were included as one of the baseline soil series characterized in Chapter 2.

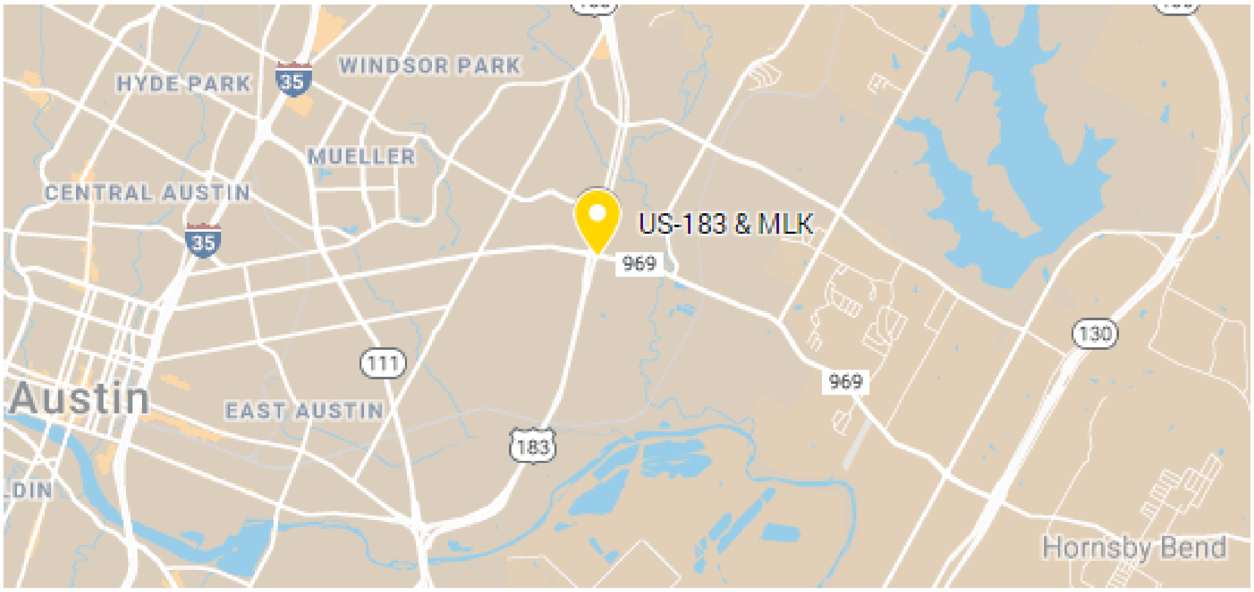


Figure 5.8.1: Site location of US 183 & MLK Jr. Blvd.

Figure 5.8.2 (a) shows the mapped soil conditions using the USDA soil survey data, while (b) shows the US Geological survey map of the area and the six locations of bucket sampling. The site is located at the top of a rise, with generally flat geologic layering beneath it. Consequently, the Taylor group is expected to be encountered beneath the Upper Colorado River deposits in this area. Soil layers identified in Figure 5.8.3 confirm this to be the case. Figure 5.8.4 shows a photograph of a hand sample recovered during sampling. This material is typical of the weathered shale material encountered during the investigation. Additionally, Figure 5.8.5 shows a gypsum seam encountered in the southbound sampling locations. Sulfate testing was performed on each bucket sample, as shown in Table 5.8.2. Because of the presence of large seams of sulfate and because of the high average concentrations even away from these seams, it was recommended that PVR treatment alternatives not include lime treatment.

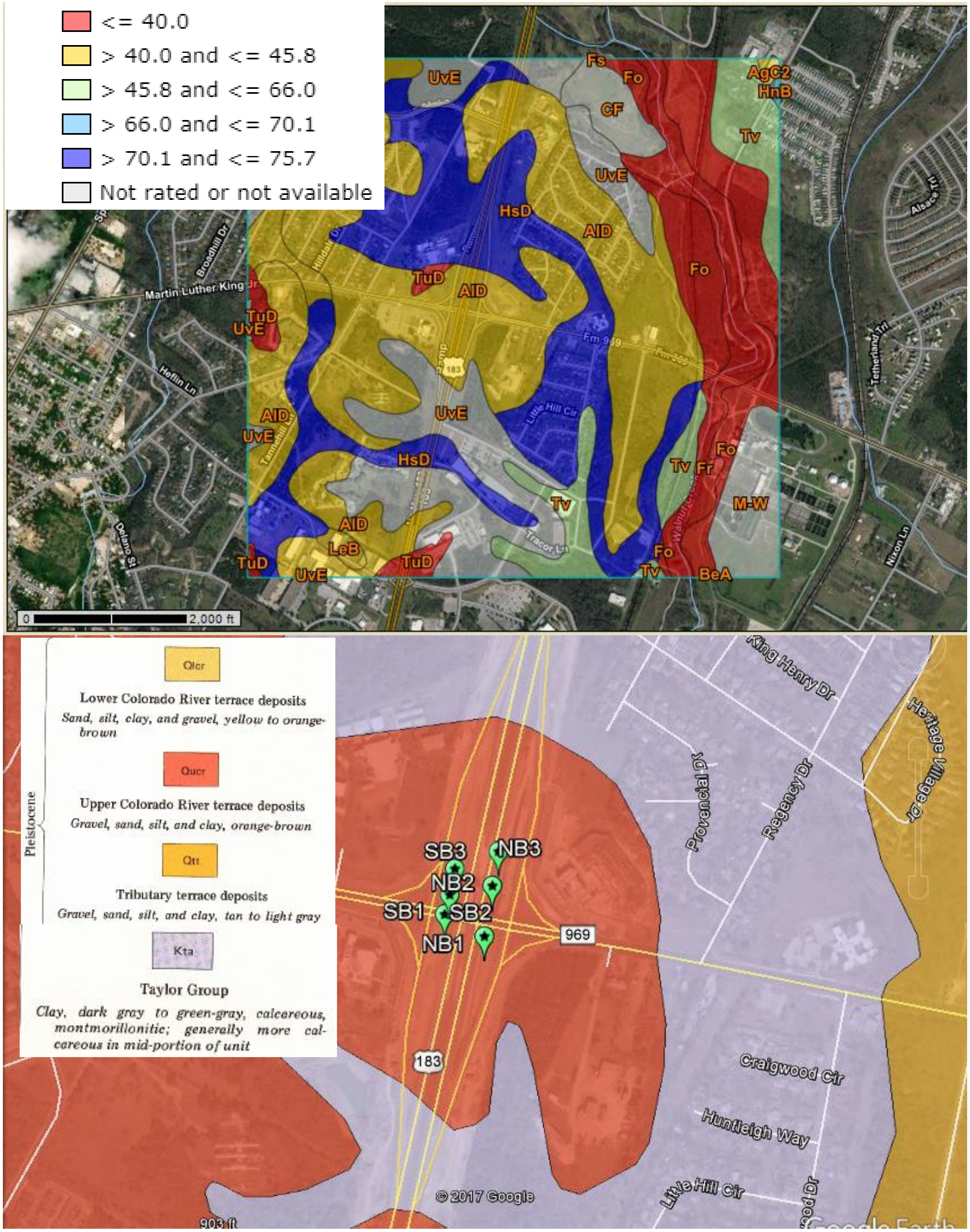


Figure 5.8.2: Sampling locations at US 183: (a) USDA; and (b) USGS maps.

Figure 5.8.3: Photo of cut from US 183 northbound, showing tan clay soils (Taylor Clay) encountered in the pavement subgrade.

Figure 5.8.4: Weathered shale sampled at northbound locations.

Figure 5.8.5: Gypsum seam identified in southbound locations.

Table 5.8.1 shows the samples collected and the characterization tests performed on soils at US 183, while Table 5.8.2 shows the results from index testing on these soils. In general, these soils can be considered moderately high plasticity clays. The PVR was calculated using data from Texas Swell Tests conducted on each bucket sample, and the calculations assume a uniform material extending to a depth of 10 ft. In only one location the PVR is considered minimal based on the highway classification.

Table 5.8.1: Table of samples retrieved from US 183.

Boring	Probable USDA Soil Type	Atterberg Limits	Grain Size	Centrifuge Testing	PVR Calculation	Soluble Sulfate Content	Lime-stabilization Characterization	PVR (Method 6048)	Site Class
SB1	N/A	X		X	X	X		4.37	High
SB2	N/A	X		X	X	X		2.18	High
SB3	N/A	X		X	X	X		4.08	High
NB1	N/A	X		X	X	X		2.34	High
NB2	N/A	X		X	X	X		4.43	High
NB3	N/A	X	X	X	X	X		0.98	Minimal

Table 5.8.2: Results from bucket samples at US 183 & MLK Jr. Blvd.

Location	LL	PL	PI	$^*\omega_{opt}$	$^*\gamma_d$ (pcf)	$^\dagger\omega_{opt}$ (SP)	$^\dagger\gamma_d$ (pcf) (SP)	TxDOT 'dry' condition	Sulfate Content (ppm)
SB1	76	20	56	27.8%	85			24%	1537
SB2	61	17	43	23.7%	94			21%	958
SB3	72	23	50	27.4%	86			23%	1140
NB1	66	19	47	25.3%	91			22%	195
NB2	71	20	51	26.5%	88	23.5%	98.2	23%	320
NB3	58	17	41	23.0%	96			21%	209
SB, average	70	20	50	26%	88				1212
NB, average	65	19	46	25%	91				241

* Predictions from NAVFAC correlation
† Measured

Grain size distributions were also performed on representative materials using a combination of wet-sieve analysis and hydrometer testing. Figure 5.8.6 shows the measured grain size distributions for selected soils at this site, noting that the 'Tan NB3' soil best represents material found in the subgrade at each of the 6 bucket locations. Figure 5.8.7 shows the standard proctor compaction curve conducted for this soil.

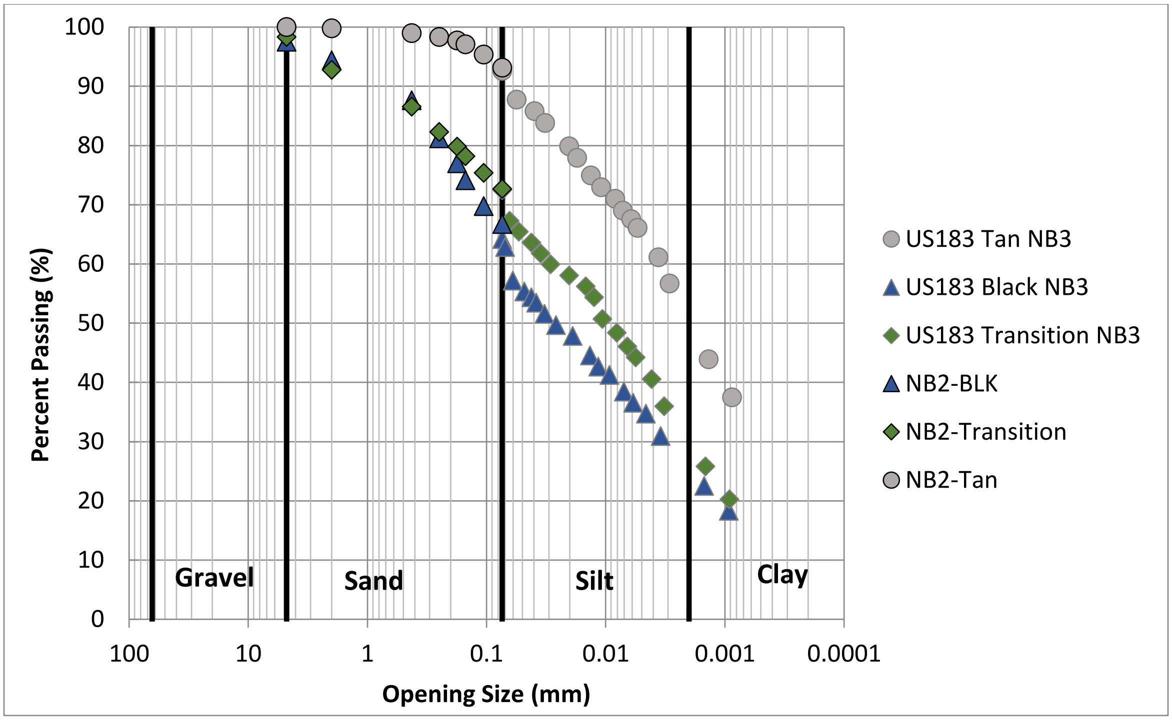


Figure 5.8.6: Grain size distributions for selected materials, showing tan subgrade soil.

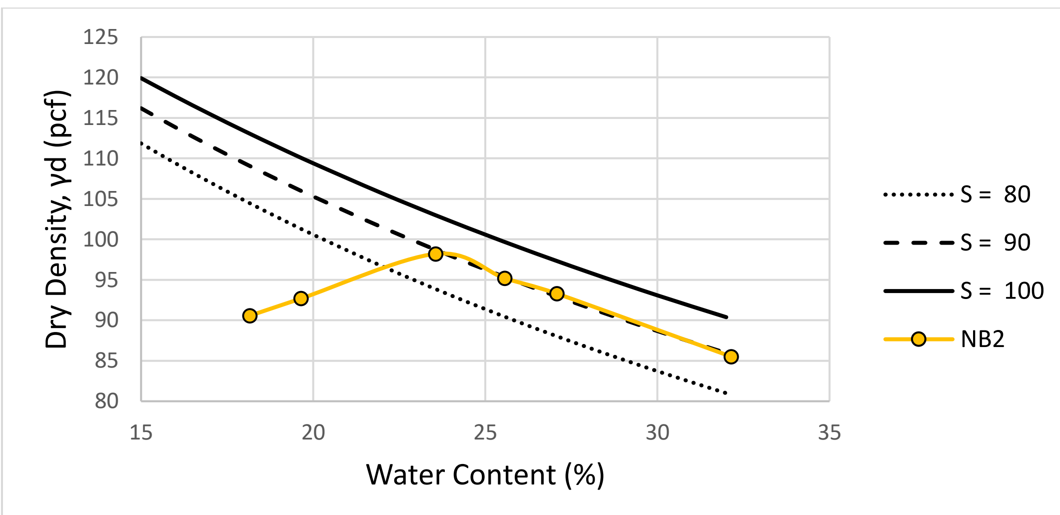


Figure 5.8.7: Proctor test data from NB2.

5.9. Driveway at FM 685

This site was identified during a highway reconstruction project along FM 685 in Hutto, Texas. The active construction provided the opportunity to place moisture sensors in the subgrade of an unpaved driveway along the project alignment, which was subsequently paved.

Figure 5.9.1 shows the site location near the intersection of US 79 and FM 685.

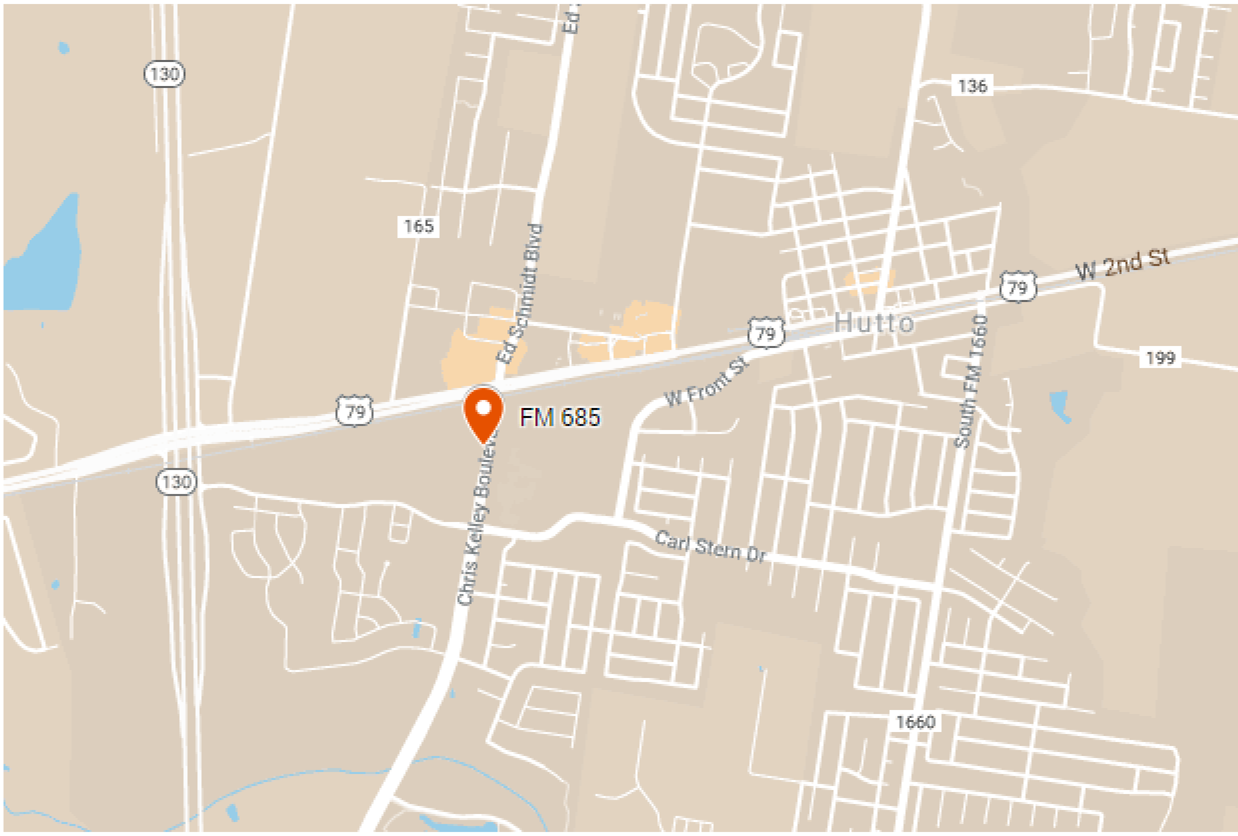


Figure 5.9.1: Site location of driveway at FM 685.

Figure 5.9.2 shows the USDA mapped soil conditions at this site, indicating that the soils are relatively uniform in the vicinity of the project. Moisture sensors were placed in the subgrade layers beneath a driveway along the highway, and surface deflections were monitored before and after paving of the driveway (Armstrong 2018). Samples of the subgrade soils were collected from this site and evaluated for PVR using Texas Swell testing.

Figure 5.9.3 shows the swell-stress curves for the different soils at this site. The upper layer at this site is composed of the Branyon clay, which is moderately expansive, while the lower layer is composed of a tan clay, which is non-expansive. Integration of these curves gives a PVR of 1.76 inches.

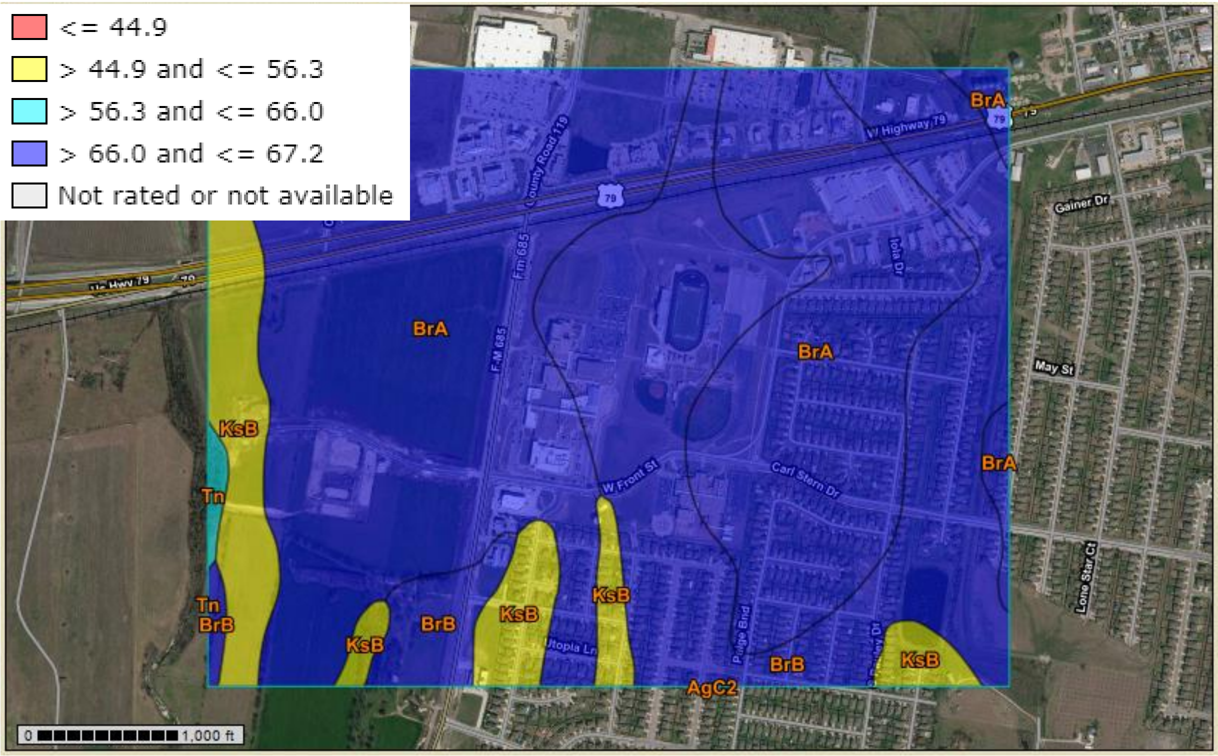


Figure 5.9.2: USDA soil survey map of FM 685 sampling location, shaded by average liquid limit from the USDA soils database.

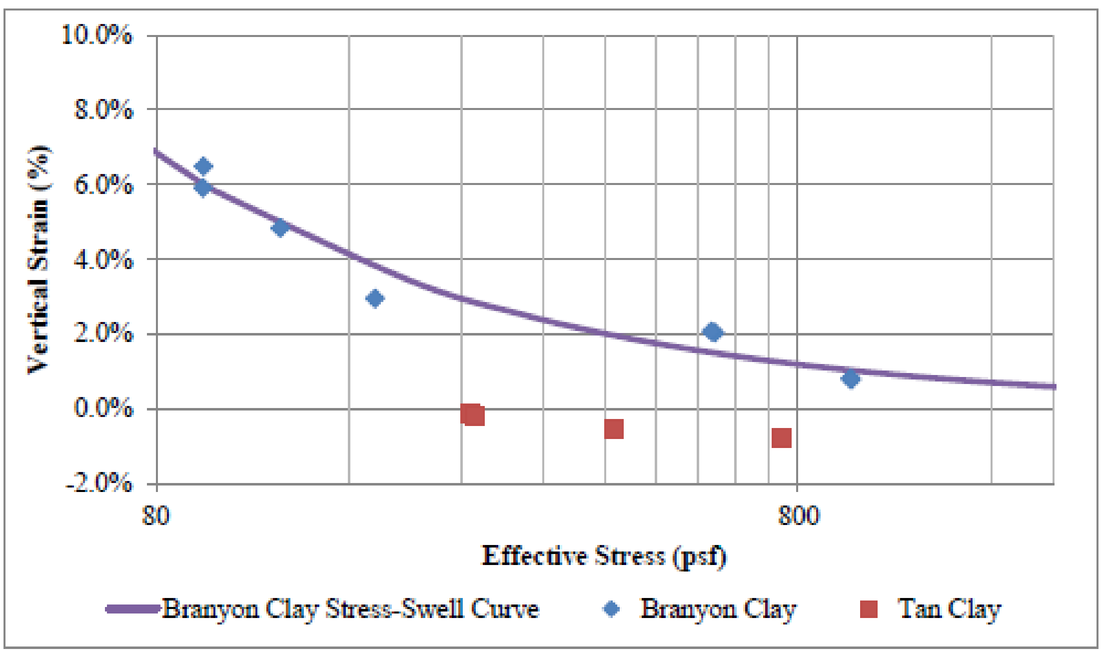


Figure 5.9.3: Swell-stress curves for FM 685 site.

Figure 5.9.4 (a) shows time histories for the moisture content and suction in the subgrade at the FM 685 driveway site, while (b) shows the daily and cumulative precipitation from a nearby weather station. The data in these figures indicate that drying in the subgrade is a slow and continuous process, corresponding to background environmental conditions, while wetting is a rapid process corresponding to rainfall events. Addition of the final impervious asphaltic pavement causes minor wetting events to have less of an impact upon the subgrade moisture, as shown in Figure 5.9.5. Figure 5.9.6 furthermore shows the impact upon subgrade moisture for comparable rainfall events before and after addition of the pavement in the driveway. Prior to addition of the final pavement, surface sensors are more likely to respond to minor rainfall events, and are more likely to respond very quickly to the addition of moisture. After paving, surface moisture sensors beneath the pavement are less likely to respond to minor rainfall events, and the rate of wetting and drying in the subgrade is slightly lower. This implies that the moisture content beneath the paved area will tend to be somewhat more stable in time. However, this data also highlights that moisture fluctuations about the average value may still be very significant, especially in the vicinity of drainage structures. Thus, the use of a PVR to describe and rank the subgrade response to environmental loadings is an appropriate simplification to this complex problem.

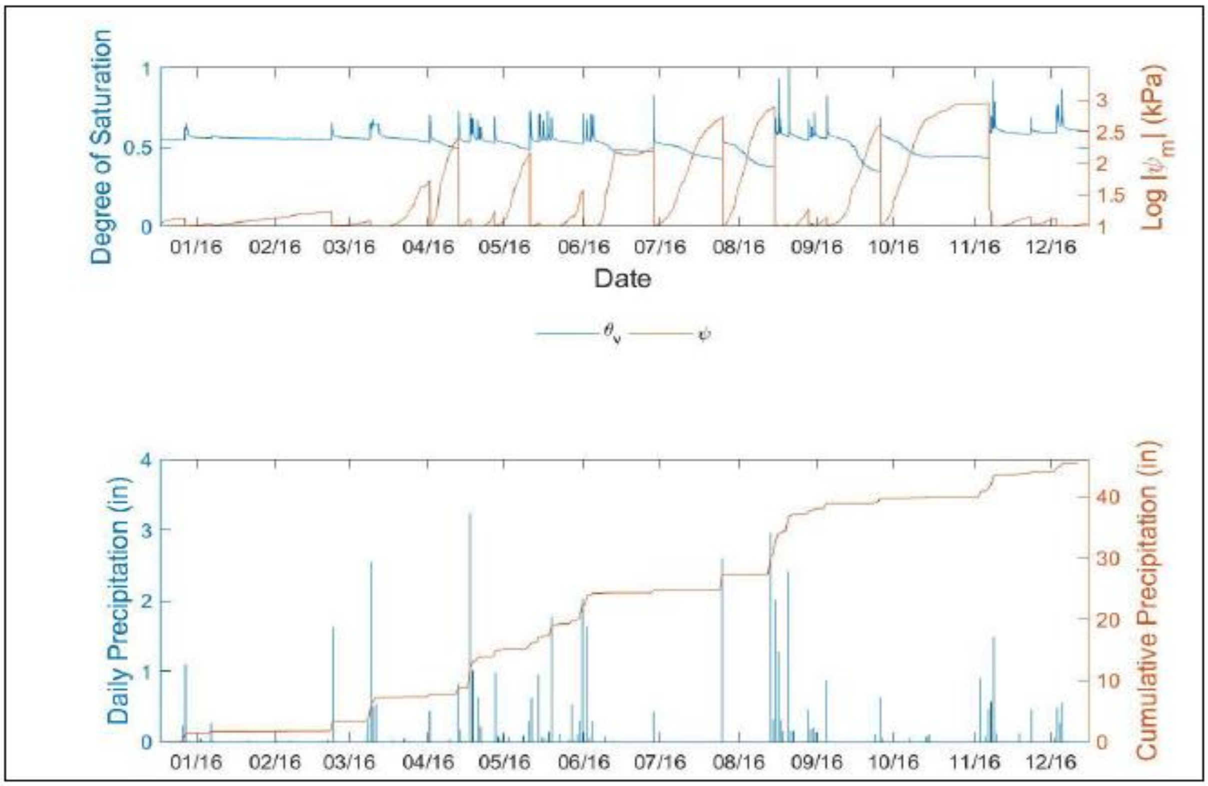


Figure 5.9.4: Time histories from the FM 685 site: (a) moisture and suction time histories; and (b) daily and cumulative precipitation data.

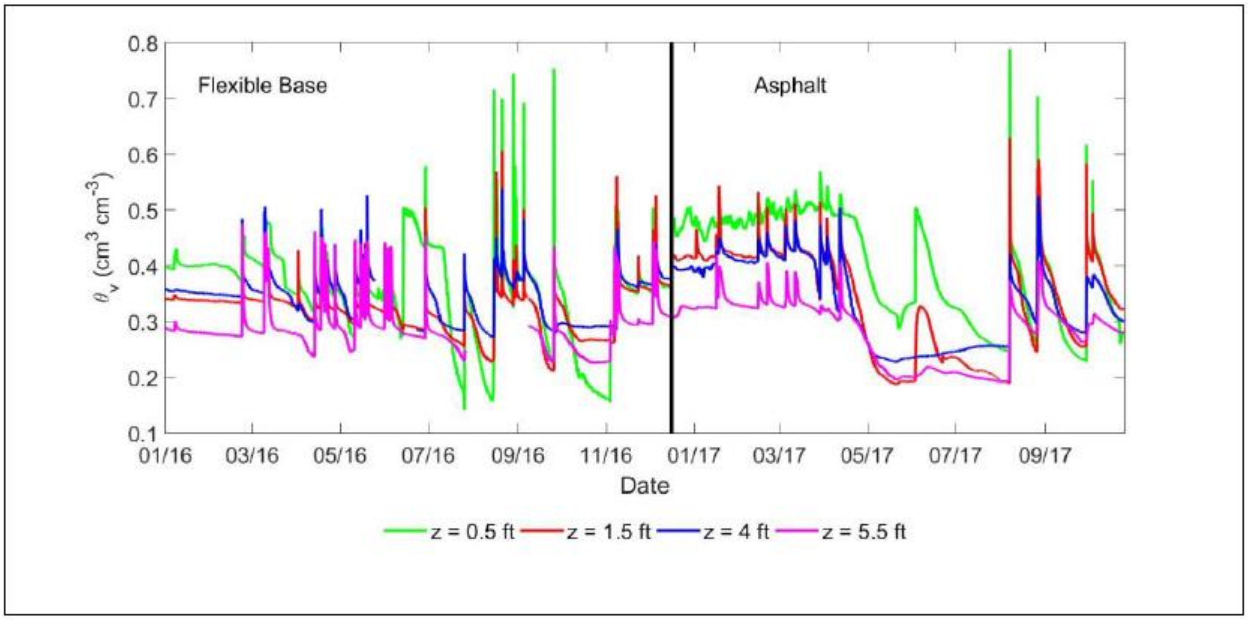


Figure 5.9.5: Subgrade moisture histories, showing time of addition of final asphaltic surface.

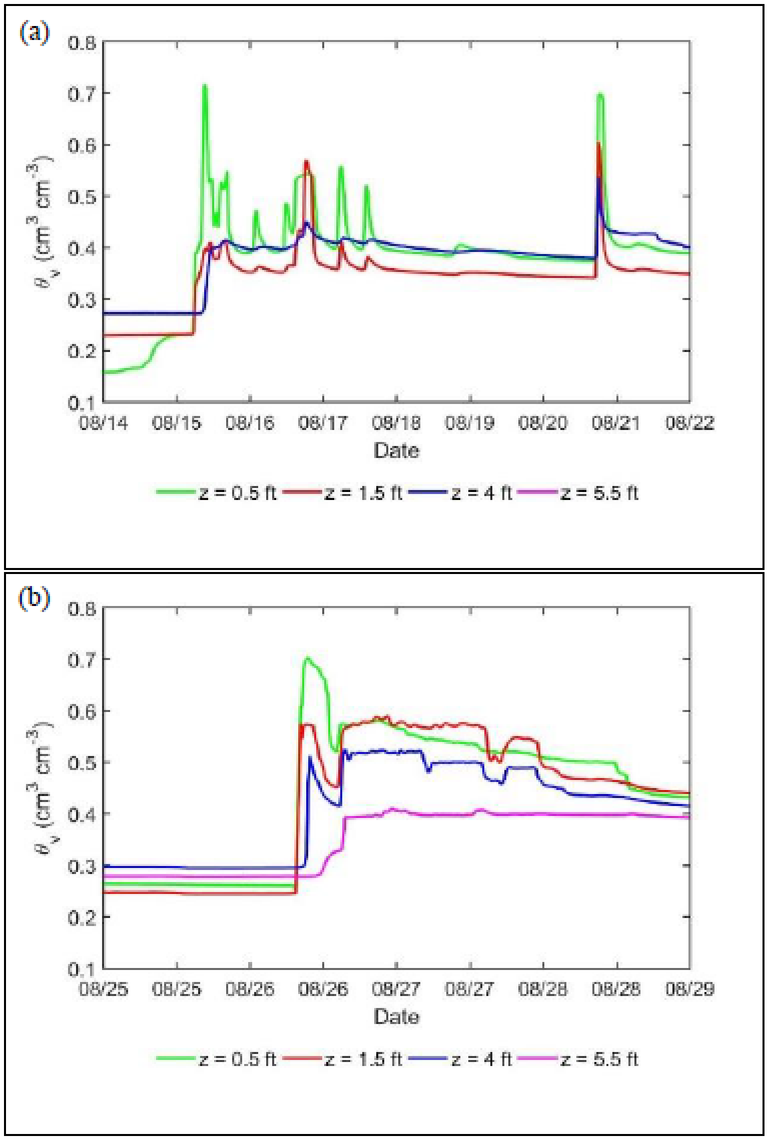


Figure 5.9.6: Moisture progression in time with depth at FM 685 site: (a) before paving; and (b) after paving, showing slower response time of moisture below paved section.

Figure 5.9.7 shows the measured vertical heave after significant wetting had taken place in the subgrade. The measured vertical heave is on the order of 10 mm, or about 0.4 inch. This is only about one-fourth of the predicted PVR of 1.76 inch. However, the actual moisture fluctuations experienced in the field soil profile may not be as wide as those experienced in the centrifuge swelling characterization test, which is intended to represent a drastic change in moisture content. Additionally, the edges of the pavement structure were found to have settled by about 10mm during the first wet season of monitoring, indicating that the pavement structure may also have been settling after construction, as well as undergoing shrink-swell cycles from moisture fluctuations.

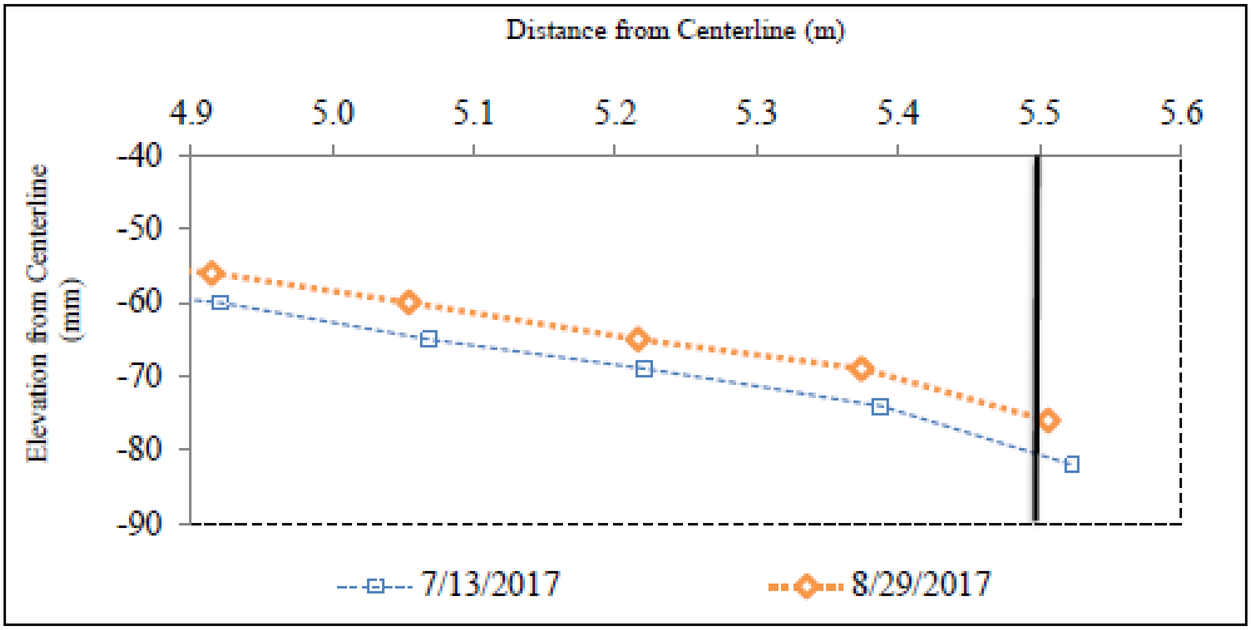


Figure 5.9.7: Pavement profiles at shoulder of FM 685 site before and after significant wetting of subgrade, showing vertical heave upon wetting.

Chapter 6. Conclusions

The conclusions drawn from each section of this report are summarized in this chapter.

Based on the data and analyses presented in Chapter 2 of this report regarding centrifuge testing on baseline soil series, the following conclusions may be drawn:

- The magnitude of swell in untreated soil depends significantly on initial conditions—moisture content and dry density. In Eagle Ford clay, an increase of moisture content by 1% was found to decrease the swell by approximately 1%. The impact of dry density on swelling is comparatively less significant; for a 3.2 pcf increase in dry density, the swelling magnitude of Eagle Ford Clay specimens was found to increase by 1%.
- For clays of lower plasticity, such as the Taylor Clay, the magnitude of these corrections is less significant, likely in correspondence with the lower expansiveness of this soil type.
- Swelling data from moisture-adjusted, trimmed samples from the field indicates that the Liquid Limit can be a good qualitative descriptor of the swelling magnitude in expansive clays without a significant coarse fraction, but that it is not a strong quantitative predictor of the swelling.
- Data generated on soils with a significant coarse fraction agrees with historical findings that replacement of the soil binder with coarse particles can reduce swelling and shrinkage magnitudes in expansive clays.
- Swell-stress curves for both natural and lime-treated soils can be reasonably approximated by a linear trend between swelling and the logarithm of stress, in a stress range from 100 to 1000 psf (or to the point of zero swell). The addition of lime to soil decreases both the slope and intercept of this swell-stress line.
- When tested under the same initial conditions and at the same effective stress, an increase in hydrated lime dosage tends to proportionally decrease the swell of a given soil sample, until the swell of the soil becomes negligible.
- Mellowing and curing of treated samples were found not to affect the swelling significantly. For the purposes of testing in the centrifuge, specimens need not be cured, and can be mellowed for 1-2 days to accommodate convenient preparation and testing schedules.

Conclusions from the data and analyses presented in Chapter 3 of this report, regarding the potential sources of variability in the centrifuge test data, are as follows:

- The magnitude of scatter in swelling that can be expected from centrifuge testing of identically-prepared soil specimens has been quantified as +/- 1.5% swelling strain. The scatter in swell potential for specimens that have identical compaction conditions is most likely due to small variations in void structure due to compaction, minor variations in mineralogy, and differences in specimen seating quality.

- The amount of vacuum grease used as a lubricant to the testing ring does not significantly affect the measured swelling. However, the application of vacuum grease to the testing ring does reduce sidewall friction for those specimens that are compacted directly into the testing rings, which is expected to aid in proper seating of the specimens. Consequently, a thin lubricating film of vacuum grease along the sides of the specimen is still recommended for use in the test procedure.

- The swelling magnitude for specimens prepared from oven-dried soil was found to be statistically less than in comparable specimens prepared from air-dried soil. While the testing program did not verify specific causes for this effect, causes may include alteration in the clay mineralogy and permanent removal of adsorbed water between the clay sheets from the heat of oven drying.

- The amount of ponded water used during the test was found to be of minor importance to the overall amount of swelling, provided that specimens remain submerged throughout the test. Consequently, the test procedure recommends adding water to the maximum fill level of each permeameter cup.

- No noticeable trend was observed in swelling with respect to local thickness variations in each specimen. Results from Section 2.2.2, however, indicate that the exact density and moisture content in each specimen will have a measureable impact upon the swelling. Consequently, specimen compaction control procedures should place higher priority on achieving a uniform *density* than on achieving a uniform *thickness*. Nevertheless, the thickness tolerance should still be controlled within ± 0.005" when possible, to minimize errors in the calculated strain values.

- Data from various methods of applying the vertical pressure during the centrifuge test indicate that the swelling results are equivalent. The precise combination of methods to achieve the target vertical effective stress should therefore be determined by the end user, though recommendations are given in the test procedure.

- The specified thickness of soil specimens was found to have a major impact upon the duration of testing, and a minor impact upon the magnitude of final swelling. Thus, deviations from the prescribed specimen thickness of 1 cm may be permitted when abnormal conditions apply to the sample, such as the presence of oversize particles in a field-trimmed sample, or when the sample is extremely expansive.

Chapter 4 presented a methodology to optimize lime treatment evaluations in expansive clays. The conclusions from this chapter include:

- Although hydrated lime is applied to moist soils in the field, a more homogeneous test sample can be obtained if lab soil samples are in an air-dry state prior to mixing with hydrated lime. This is expected to reduce scatter in the prediction of the treatment dosage.

- For the highly-expansive Eagle Ford Clay, the swell pressure extrapolated from the swell-stress data using various curve-fitting models was found to show relevant scatter. In some cases, the swell pressure predicted in this manner was higher for treated soils than for untreated ones, due to scatter in the measured swelling values.

- Based on the variability in obtaining the swell pressure, the reduced procedure relies instead on a reduction in slope of the swell-stress curve as the means of interpreting the effect of lime treatment.

- Thus, for the purposes of calculating PVR for lime-treated soils, the approximation of a constant swell pressure appears to be adequate. The assumption of a unique swell pressure for both untreated and treated samples of a given soil allows for testing protocols that can expedite the testing program for practical purposes.

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Appendix A: Testing Protocols for the Texas Swell Test

Test Procedure for

SWELLING OF EXPANSIVE CLAYS (TEXAS SWELL TEST)

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1. SCOPE

1.1. The swelling of expansive clays can be characterized by defining the soil swell curve, which establishes the relationship between magnitude of swelling and confining pressure. This document includes the procedures for sample preparation, sample conditioning and testing to be used in the Texas Swell Test.

1.2. The sample preparation methods that can be adopted include those for undisturbed samples (sample trimming) and reconstituted samples (by semi-static or kneading compaction)

1.3. The sample conditioning methods, or pre-treatment methods, that can be considered include moisture conditioning, treatment with hydrated lime, and moisture-adjustment on previously compacted samples

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2. DEFINITIONS

2.1. *Expansive Soils*—soils which contain enough fine material of high enough plasticity such that drying or wetting can produce significant volume change

2.2. *Potential Vertical Rise*—Potential Vertical Rise, expressed in mm (in.), is the latent or potential ability of a soil material to swell, at a given density, moisture, and loading condition, when exposed to capillary or surface water, and thereby increase the elevation of its upper surface, along with anything resting on it.

2.3. *Liquid Limit*—A liquid limit (LL) is the moisture content expressed as a percentage of the weight of oven-dried soil, at which soil changes from a plastic to a liquid state. It is the moisture content of a soil at which two halves of a soil part, separated by a groove of standard dimension (1 cm deep), will join at the length of 1/2 in. under impact of 25 blows using the Mechanical Liquid Limit Device, and Tex-104-E. The percent of moisture in a soil sample where a decrease in moisture changes from a viscous or liquid state to a plastic state.

2.4. *Plasticity Index*—Plasticity index is a test conducted on soil samples as set out in Tex-106-E. The plasticity index is a range of moisture in which a soil remains in a plastic state, while passing from a semisolid state to liquid state. Numerical difference between Liquid Limit and Plastic Limit of a soil (PI = LL - PL) using Tex-106-E.

2.5. *Overburden*—The overburden is the soil above the layer or layers being investigated. Example: A clay layer covered with 3.1 m (10 ft.) of sand would have 3.1 m (10 ft.) of overburden on it.

2.6. *G-level*—a linearized multiple of Earth gravity which is imposed by rotation about central axis, as in a centrifuge

2.7. *Moisture conditioning*—the addition of water to a soil in a dry or powdered state in order to achieve a target water content

2.8. *Moisture adjusting*—the addition or removal of water into/from a soil sample (usually under undisturbed conditions) by placement of the soil sample in a controlled environmental chamber until reaching the target moisture content

NOTE 1: Because swelling in soils occurs as the soil moves from a comparatively dry to a wet state, the field (in-situ) moisture content and density may not be representative of dry conditions if sampling takes place during a wet season.

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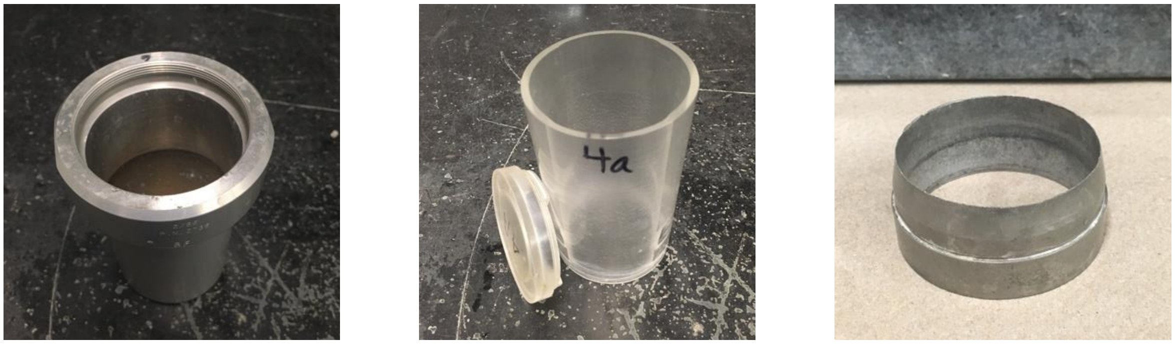
3. APPARATUS

3.1. *Hydraulic Centrifuge*, capable of reaching accelerations of at least 250 G's and with a rotor capable of testing 6 samples concurrently. Centrifuge should be outfitted with in-flight data acquisition system and linear position sensors to continuously monitor sample height during testing.

3.2. *Metal Centrifuge Buckets,* 6 units.

3.3. *Centrifuge Permeameter Cups with Threaded Base,* 6 units. Cups maintain a clearance fit into centrifuge buckets, and material used to manufacture the cups should have a Young's Modulus exceeding 71×10^3 psi (e.g., acrylic). Fig. 1 shows examples of these components.

3.4. *Cutting Rings, 2" diameter,* 6 units. Rings should fit snugly into permeameter cups.



(a) (b) (c)

Figure 1: View of components of the Texas swell centrifuge used to host the soil sample: (a) centrifuge bucket; (b) permeameter cup; and (c) cutting ring.

3.5. *Filter Paper*, 12 sheets, trimmed to 2” diameter circles.

3.6. *Brass Porous Discs*, 6 units, with thickness of 0.125” and a nominal diameter of 2”.

3.7. *Brass Porous Discs*, *6 units*, with thickness of 0.1” to 0.4” and diameter of 2”. It is recommended that, for swell testing of natural (i.e. untreated) soil samples, 2 porous disc of each height be used. Instead, for swell testing of lime-treated soil samples, the 0.2” porous discs be used. Fig. 2 shows examples of these components.

Fig. 2. Left to right: 0.1”; 0.2”; and 0.4” brass porous discs.

3.8. *Compaction* weight

3.9. *Rubber* mallet

3.10. *Small kneading* compaction *hammer*

3.11. *Syringes*, 6 units, 100 mL capacity.

3.12. *Bowl*

3.13. *Spray Bottle*, at least 250 mL capacity.

3.14. *Metal spatula*

3.15. *Moisture Content Trays*

3.16. *Balance,* capable of reading in increments of 0.01gram up to 1,000 grams.

3.17. *Vertical Caliper*, capable of reading in increments of 0.001” up to 2.000”.

3.18. *Drying Oven,* capable of continuously heating at 110 +/- 5 °C.

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4. MATERIALS

4.1. Approximately250g air-dried, processed soil for each set of 6 samples

4.2. Water

*4.3. Hydrated Lime (*for testing programs involving lime treated specimens)

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5. SUMMARY OF OVERALL TESTING PROCEDURE

5.1. A minimum set of physical measurements required by the test is described in Section 8

5.2. Soil samples should be prepared according to the specified dry density and water content, along with any chemical treatment required, as outlined in Sections 6 and 7.

5.3. The components of the centrifuge test should be weighed, balanced, and installed in the centrifuge apparatus, referring to the steps outlined in Section 8.

5.4. Centrifuge testing speeds should be selected in accordance with the desired overburden stress range. Guidance on this process is contained in Sections 8.1 and 8.2, although the precise values will be machine-specific, and should be calibrated prior to use.

5.5. A minimum of five data points should be recorded after placing samples in the centrifuge and before starting the centrifuge

5.6. A seating load of 5-g should be applied for 3 to 5 minutes.

5.7. After reaching the target speed, the samples should be allowed to compress (“dry consolidation”) for not less than 30 minutes or until the heights stabilize sufficiently, but in no case should dry compression exceed 2 hours if the permeameter cups are open to atmospheric drying.

5.8. After the compression stage, the centrifuge should be stopped and water added using calibrated syringes. Care should be taken to perform this operation in a minimum amount of time so as to minimize the amount of swelling which may occur under 1-g loading.

5.9. The centrifuge should be restarted, and samples allowed to swell until reaching a clear reduction in swelling rate (as shown in a plot of vertical strain versus the logarithm of time). The swelling under a reduced rate has been commonly referred to as secondary swelling. A minimum testing time of not less than 8 hours is recommended, even for soils that do not appear to swell significantly during the initial few hours of testing.

5.10. After reaching secondary swelling, the centrifuge should be stopped, and the samples should be allowed to rebound for a minimum of 1 hour while still recording data. Longer rebound times can be adopted, as it may add accuracy to the measurement of final water content. In addition, the full rebound characteristics of the sample at very small loads may be facilitated with additional rebound time.

5.11. After completion of the rebound stage, samples should be extracted from the testing cups, any excess water should be removed from the testing ring and faces of the sample, and the water content should be determined. Care must be taken to minimize the time between removing samples from the centrifuge and the final removal of ponded water, as some additional moisture may be absorbed during this procedure.

5.12. Additional stages of air-drying shrinkage are possible immediately after test completion (and before measuring the final water content). However, these approaches are not covered in this document.

6. INITIAL CONDITIONS AND SOIL SAMPLE CONDITIONING

6.1. The centrifuge PVR methodology requires that samples be compacted to the same target density and moisture content values for comparison purposes. In principle, the initial water content for testing should be chosen as being consistent with the moisture content (or soil suction) expected during the driest condition in-situ during the project design life.

Once the moisture content is selected, the dry density should be chosen so that the soil structure is consistent with in-situ conditions.

6.2. Determination of initial soil sample conditions using a target initial Saturation: A simple relationship to define the initial soil sample testing conditions is the moisture content corresponding to the "Dry" condition, w_{dry}, in TxDOT standard TEX-124-E:

$$w_{dry} = 0.2 * LL + 9 \tag{1}$$

where LL is the soil liquid limit (in percent).

6.3. The dry density may be calculated using the following relationship, assuming a constant degree of saturation:

$$\gamma_d = \frac{G_s}{1 + G_s * \frac{w}{S_r}} * 62.4\, pcf \tag{2}$$

where Gs is the soil specific gravity (dimensionless), w is the precise moisture content of the soil after it has been prepared to the target moisture condition, and S_r is the degree of saturation.

S_r values ranging from 0.85 to 0.9 have been found to match data from soil shrinkage tests performed on fine-grained samples within the range of water contents predicted by Equation (1).

NOTE 2: Despite the relatively high degree of saturation, the water content corresponding to Equation (1) has been shown to result in soil samples with a reasonably high suction value after compaction, and containing only relatively few macro-voids. These conditions are expected to lead to soil samples that are representative of in-situ soils having undergone natural drying shrinkage in the field.

NOTE 3: For samples with a significant coarse fraction or with water content values smaller than those resulting from Equation (1), the densities specified by Equation (2) may be unrealistically large. In this case, a different approach may be needed to define the initial density of the soil sample.

6.4. Determination of initial soil sample conditions using moisture content derived from Compaction Curves: An alternative approach to define the target initial conditions involves selecting a density corresponding to $\gamma_{d,max}$ and a moisture content corresponding to w_{opt} - 3% , where $\gamma_{d,max}$ is the maximum dry unit and w_{opt} is the optimum moisture content as defined from Standard Proctor tests. Because of constrains in time and available soil to conduct Standard Proctor Tests for different soils in a given profile, the target densities and moisture contents may be defined using correlations with the Atterberg limits, as determined by USACE Correlations documented in the Construction Control for Earth and Rock-Fill Dams Engineering Manual (USACE, 1995). The correlations for optimum moisture content and maximum dry unit weight are:

$$w_{opt} = 0.24LL + 7.349 \quad (3)$$
$$w_{target} = w_{opt} - 3 = 0.24LL + 4.349 \quad (4)$$
$$\gamma_{d,max} = -0.414LL + 123.704\ (pcf) \quad (5)$$
$$\gamma_{target} = \gamma_{d,max} \quad (6)$$

6.5. Soil Processing: Material to be used in reconstituted samples should be air-dried and processed so that the maximum particle size is less than ½ the sample thickness. The relative proportions of binder and the coarse fraction as in the original sample should be maintained whenever possible.

6.6. Moisture conditioning: Moisture conditioning is performed by adding water to the air-dried soil to reach the target initial moisture content . It may be necessary to measure the moisture content of the air-dried powder prior to computing the amount of moisture to add to the processed sample.

Air-dried, processed soil should be mixed to the volume of water needed to achieve the target moisture content and allowed to rest for 12-24 hours before measuring moisture content. The required amount of water to add to the air-dried soil, in order to reach the target moisture content, can be defined as follows:

$$m_{w,req} = \frac{m(w_{target} - w_{AD})}{1 + w_{AD}} \quad (7)$$

6.7. Moisture Adjusting: In case undisturbed soil samples are to be tested, the in-situ moisture content should be initially determined. If the in-situ moisture is higher than the target initial value, the sample moisture content should be adjusted in a controlled manner. This may be achieved by placing the soil sample a controlled relative humidity chamber or controlled rate-of-drying chamber.

The weight of the undisturbed sample being moisture adjusted should be checked periodically to determine when the target moisture content has been achieved.

6.8. Chemical Treatment: Chemical treatment of soil samples can be performed in order to prepare soil specimens representative of treatment characteristics (e.g. treatment type, treatment dosages) and methods to be adopted in the field. When using chemical stabilizers which require mixing, it is recommended to mix the soils thoroughly so as to achieve homogeneity of the treatment in the samples.

A useful testing program involves preparation of multiple soil specimens treated using several lime dosages, and testing them in order to define the impact of the treatment on swelling and, consequently, on the PVR. This is because data from such testing program can be directly used in a PVR evaluation for different treatment depths and dosages.

Hydrated lime may be added to the soil either as a powder or as a slurry. In either case, lime dosage, the moisture content of the final mixture (before hydration reactions occur) should correspond to the target moisture content for testing.

To prepare lime-treated soil specimens, hydrated lime should be added at a dosage defined as a percentage of the mass of soil solids. The mass of hydrated lime to be added can be predicted as follows:

$$m_{HL} = \frac{HL_{target} * m}{1 + w_{AD}} \quad (8)$$

where m_{HL} is the mass of hydrated lime, HL_{target} is the target mass percentage of hydrated lime, m is the mass of air-dry soil and w_{AD} is The gravimetric water content of the air-dry soil.

The appropriate amount of hydrated lime should be mixed with air-dried soil before moisture conditioning to achieve a homogenous mixture. The mass of solid lime added to the air-dry soil should be accounted for in the calculation of the moisture content. The required amount of water for moisture conditioning of lime-treated specimens can be defined as follows:

$$m_{w,req} = \frac{m(w_{target} - w_{AD})}{1 + w_{AD}} + w_{target} m_{HL} \quad (9)$$

Where $m_{w,req}$ is the mass of water required to achieve the target moisture content

m is the mass of air-dry soil prior to adding in Lime solids

w_{target} is the target moisture content

w_{AD} is the air-dry moisture content

and m_{HL} is the mass of lime solids added to the mixture

As with untreated moisture-conditioned soil, lime-treated moisture-conditioned soil should be allowed to rest for 12-24 hours before preparing test samples. As with untreated samples, the water content of a representative quantity of the mixture should be measured directly after mixing in order to define any adjustment needed to achieve the target density.

7. SAMPLE PREPARATION AND COMPACTION PROCEDURES

7.1. Given the target dry density (in pcf) and moisture content (as a decimal) of the samples, the required mass of soil for each sample may be calculated as follows.

$$m_{req} = \gamma_{d,target}(1 + w_{target}) * \frac{1\,{}^{g}/_{cm^3}}{62.4\,pcf} * A_{sample} * h_{target} \quad (10)$$

The target height of the soil sample should be approximately 1.00 cm (0.393 in), although it may be adjusted within a range of 1.5 cm to 0.8 cm (0.6 to 0.3 inch) to suit the needs of the project. . The diameter of the testing rings should be 5.08 cm (2 in), resulting in a cross sectional area of approximately 20.27 cm^2. (3.14 sq.in.)

7.2. Compaction procedures involve a 2-inch diameter mold, which may also be used as the testing ring.

Figure 3 shows an example set of equipment necessary to perform methods A and B of this procedure.

(a) (b)

Figure 3: Typical components for soil sample preparation: (a) compaction mold; and (b) loading apparatus.

NOTE: Unless otherwise indicated, mass measurements should be conducted to the nearest 0.01gram and all height measurements should be determined to the nearest 0.001 inch.

7.3. Compaction Method A (Kneading Compaction): Kneading compaction can be adopted, particularly if it necessary to produce a samples at particularly high density. It may also be needed for cases involving samples with significant fraction of granular or organic materials.

Two alternative kneading compaction approaches can be adopted: Method A.1, in which samples are transferred from the compaction mold to a testing ring prior to testing, and Method A.2 in which samples are tested directly in the compaction ring.

Method A.1 minimizes the side-wall stresses under initial testing conditions, which may allow better seating of the sample, and may represent better the in-situ stress conditions of expansive soil deposits that have undergone moderate drying. Also, if Method A.1 is used, samples could be stored (generally no more than to 24 hours) prior to testing, provided that no moisture is gained or lost by the samples during this period.

Method A.2 may be required when the sample is at risk of disintegration or other damage during the transfer from the compaction mold to the testing ring.

The specific steps to be followed when adopting Compaction Method A (kneading compaction are detailed below).

Step		
1.	Calculate the initial moisture content and target dry density	
2.	Calculate the soil mass to prepare the sample	
3.	Assemble the compaction mold	
4.	Place a sheet of protective filter paper at the bottom of the mold	
5.	Weigh the moist soil to the nearest 0.01g and add to the compaction mold	
6.	Level the soil and lightly tamp to minimize the loss of any loose particles	
7.	Compact the sample using a small compaction hammer operated by hand, taking care to maintain an even compaction pattern	
8.	A comparatively larger foot may be used in conjunction with a mallet in order to achieve a comparatively flat surface and higher compactive effort	

9.	Measure specimen height regularly throughout the compaction process using a dial indicator. Repeat until the specimen has been compacted within 0.005” of the target height.	
10.	If samples are to be tested within the <u>compaction</u> ring (Method A.2), skip to Step 14	
11.	Extrude samples from mold. If samples should be trimmed into testing rings immediately, proceed to Step 13	
12.	If samples will be stored overnight prior to testing, wrap each sample in plastic wrap and place specimen in an air-tight plastic bag, making sure that any excess air has been removed.	
13.	Prior to testing, trim samples into a pre-weighed and lightly-greased cutting ring	
14.	Place top and bottom disk in contact with the sample, protecting the disks with filter paper. Ensure that disks are well-seated on the sample	

7.4. Compaction Method B (Static Compaction): Static compaction may be used to expeditiously produce a uniform sample at comparatively low moisture contents and high density, particularly when the expansive clay to be tested contains little or no coarse fraction.

The use of mechanical stops in the static compaction mold greatly facilitates the preparation of uniform soil specimens.

Two alternative static compaction approaches can be adopted: Method B.1 in which samples are transferred from the compaction mold to the testing rings after compaction, and Method B.2 in which samples are tested directly in the compaction rings.

Method B.1 minimizes the side-wall stresses under initial testing conditions, which may allow better seating of the sample, and may represent better the in-situ stress conditions of expansive soil deposits that have undergone moderate drying. Also, if Method A.1 is used, samples could be stored (generally no more than to 24 hours) prior to testing, provided that no moisture is gained or lost by the samples during this period.

Method B.2 may be required when the sample is at risk of disintegration or other damage during the transfer from the compaction mold to the testing ring.

Step		
1.	Calculate the initial moisture content and target dry density	
2.	Calculate the mass of soil to add	
3.	Assemble the compaction mold	
4.	Place a sheet of protective filter paper inside the bottom of the mold	
5.	Weigh out the moist soil to the nearest 0.01g and add to the compaction mold	
6.	Level the soil and lightly tamp to prevent the loss of any loose particles	
7.	A single piece of filter paper cut to size should be placed above the loose soil to aid in separating the sample from the mold	

8.	Place the compaction sizing disk and compression block above the soil sample and pressed down evenly.	 
9.	The sample is compacted using a hydraulic jack or other means of applying static pressures to the sample. Compaction proceeds until limited by the available load or by the limits of the mold. Care should be taken not to continue to load the mold once the mechanical limits are reached, as the edges of the mold may buckle under very high normal loads. Continue until the sample has been compacted within 0.005” of the target height	
10.	If samples are to be tested inside the compaction ring (Method B.2), skip to Step 14	
11.	Extrude samples from mold. If samples are to be trimmed into testing rings immediately, proceed to Step 13	
12.	If samples are to be stored overnight prior to testing, wrap each sample in plastic wrap and place in an air-tight plastic bag, taking care to remove any excess air	
13.	Prior to testing, trim samples into pre-weighed lightly-greased cutting ring	
14.	Place top and bottom disks in contact with the sample, protecting the disks with filter paper. Ensure that the disks are is well-seated on the sample	

8. TESTING PROCEDURE

8.1. The target G-level for the test should be selected so that the effective stress during testing is consistent with the range of stresses at the depth that the sample was collected. Table 1 provides G-level recommendations to reach appropriate stress ranges. The values listed in the figure assume that: (1) The centrifuge has a radius of operation equal to 0.22 m, and (2) The three aforementioned brass porous discs are used to impart overburden stresses.

Table 1: Selection of centrifuge g-levels for testing

APPROXIMATE FIELD STRESS				
Assumptions:				
Total Unit weight of soil	120 (pcf)			
Overburden at surface	150 (psf)			
Depth Interval	Depth		Desired Stress	
(ft)	(ft)	(ft)	(psf)	(psf)
0-2	0	2	150	390
2-4	2	4	390	630
4-6	4	6	630	870
6-8	6	8	870	1110
8-10	8	10	1110	1350

APPROXIMATE CENTRIFUGE STRESS			
Standard G-level	52		
Standard Radius	21.7 (cm)		
	A: Low Overburden (Small Disk)	B: Medium Overburden (Medium Disk)	C: High Overburden (Large Disk)
Stress at 52 g's ->	350	500	820
G-level	(psf)	(psf)	(psf)
20	135	192	315
32	215	308	505
50	337	481	788
65	438	625	1025
85	572	817	1340

8.2. The selection of different testing stress levels can be selected to optimize the number of specimens to be tested in a given spin of the Texas Swell Centrifuge. This is because the equipment has the capability of testing six specimens in a given Spin. Figure 4 shows Standard testing plans, which allow the user to optimize the generation of the data.

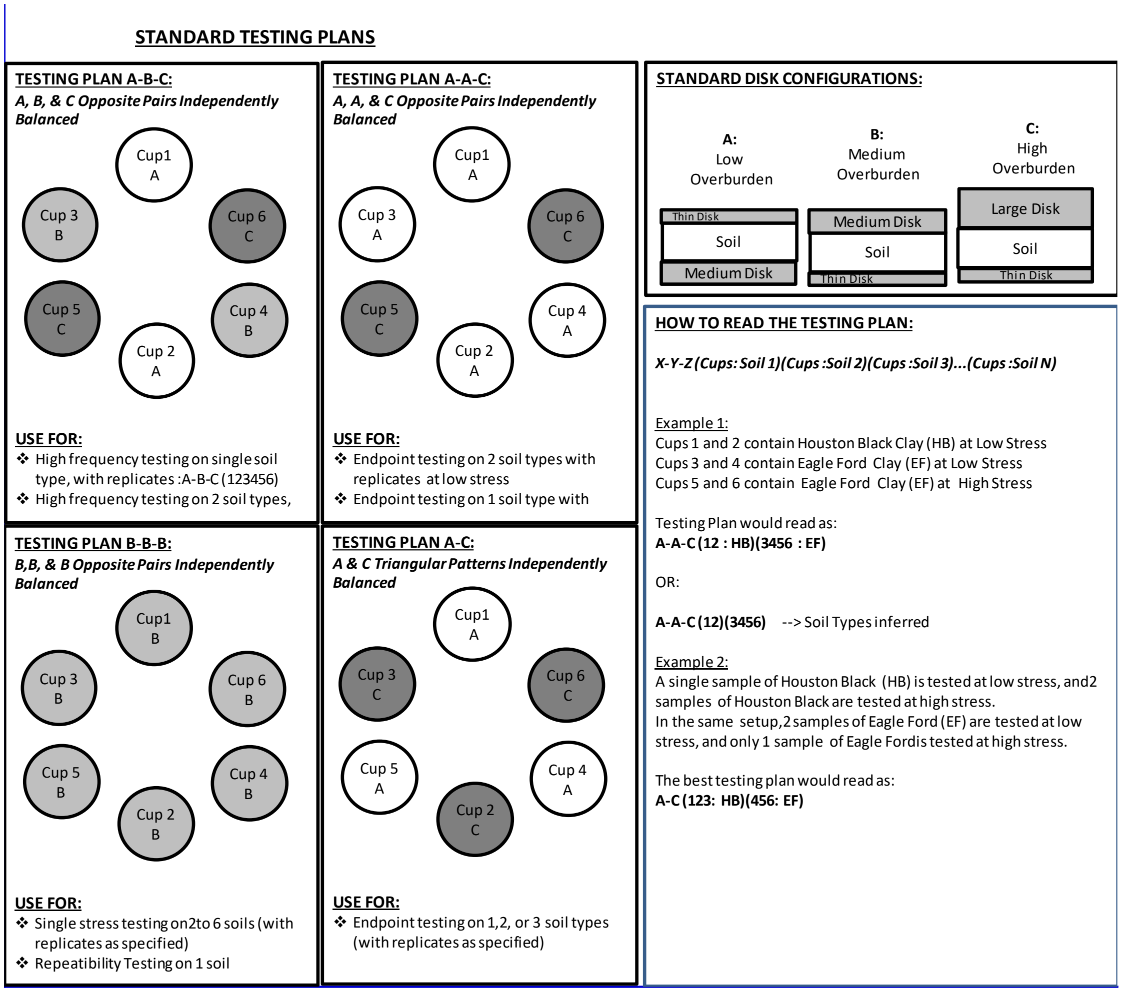
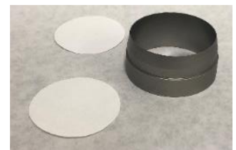
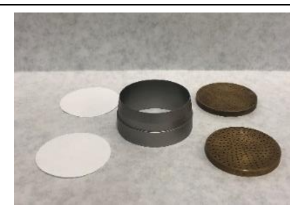


Figure 4: Proposed testing plans to optimize data generation.

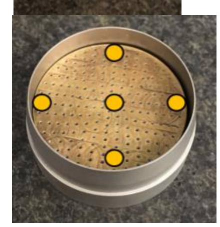
8.3. The steps required to set up a centrifuge test are indicated below. They follow closely the datasheet provided with the spreadsheet program used to reduce the data. This datasheet can be found in *Appendix A: Example Testing Datasheet.*

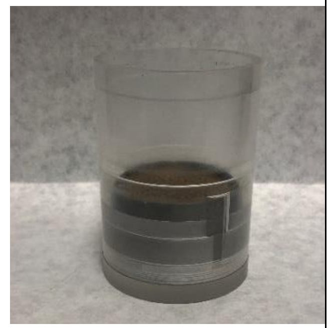
8.4. Pre-test setup:

1.	*Mass (cup)*	Record the mass of the entire cup to the nearest 0.01 g	
2.	*Mass (ring, grease, filter paper)*	Record the mass of the ring, grease, and filter paper to the nearest 0.01 g Top and bottom filter papers for each sample should be trimmed so that they fit cleanly inside the cutting ring. Each cutting ring should be greased sparingly with vacuum grease to reduce the side friction on the samples, but there should not be an excess of grease on the rings.	
3.	*Mass (r+g+fps+disks)*	Record the mass of the ring, grease, filter papers, and top and bottom disks to the nearest 0.01 g This value is used to check the actual mass of soil used in the centrifuge test	
4.	Mass (1 fp, t. disk)	Record the mass of one filter paper and the top disk This value will be used in the computation of the overburden stress	
5.	Height (1 fp, t. disk)	Record the height of a single piece of filter paper and the top disk to the nearest 0.001 inch This value (along with that in Step 6) will be subtracted from the final height measurement to provide the thickness of the sample	

6.	Height(1 fp, b.d)	Record the height of a single piece of filter paper and the bottom disk to the nearest 0.001 inch This value will be used to adjust the bottom radius of operation for the centrifuge sample, as well as in the computation of the thickness of the soil sample as compacted	

8.5. Additional height measurements should be made after specimen compaction, as follows:

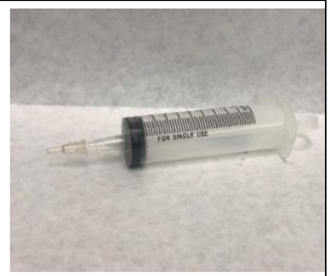
7.	*Sample Heights*	Place the filter papers, top disk, and bottom disk in contact with the sample inside the greased testing ring	
	Height @ Center	Measure and record the height of the sample and both disks at 5 locations. This value will be used to calculate the actual thickness of each centrifuge sample	 
	Height @ 0		
	Height @ 90		
	Height @ 180		
	Height @ 270		
8.	*Mass (ring+soil)*	Record the mass of the testing ring, soil, disks and filter papers This value will be used in subsequent phase calculations for the sample	

9.	*Weight (cup assembly)*	Record the mass of the entire cup assembly	
10.	*Weight (Centr. Bucket+Cup)*	Record the mass of the centrifuge bucket and cup assembly both for the purposes of centrifuge balancing and for later use in the back-calculation of the height of ponded water after the test. Additional mass may be needed in the bottom of the centrifuge bucket for balancing purposes To ensure that centrifuge is balanced, each total apparatus (permeameter and centrifuge bucket) should be weighed and the total mass should be compared for each balanced set. Balanced sets should have the same total mass within 1 g. If the difference in mass exceeds this, the lighter samples in the set should be augmented with extra mass at the bottom of the centrifuge bucket.	 
11.	*Extra Spacer thickness*	Record any additional height contributed to the system by the addition of the balancing mass.	

8.6. The soil specimens should be placed in the Texas Swell C Centrifuge as follows:

1.	*Loading the Centrifuge*	Each centrifuge bucket hosting a soil specimen should be placed in the appropriate position on the centrifuge rotor and the lid should be properly secured. Ensure that wires are free and that both the lid and linear position sensor are fit snugly, to minimize any movement during testing that may affect the sensor readings	
2.	*Initial Readings*	After the specimens are placed into the centrifuge, the data acquisition system and then the centrifuge should be started (see appropriate User Manual for details). Initial sensor readings should be collected prior to starting the centrifuge motor.	
3.	*Seating Load*	The samples should be allowed to compress under a seating load of 3 to 5 G's for approximately 5 minutes	
4.	*Dry Consolidation*	After the seating phase, the specimens should be spun to the target G-level for approximately 30 minutes, or until sample height stabilizes. If stoppers are not used in the moisture ports, the dry consolidation stage should not exceed 2 hours.	
5.	*Addition of Water*	After the dry consolidation phase, the centrifuge should be stopped, and the water added to the moisture ports using calibrated syringes. Care should be taken not to move the LVDT sensor, as this may affect the swell readings. Care should also be taken to expedite this process, in order to minimize the amount of swelling occurring under 1-G loading conditions.	

8.7. Addition of water to centrifuge samples:

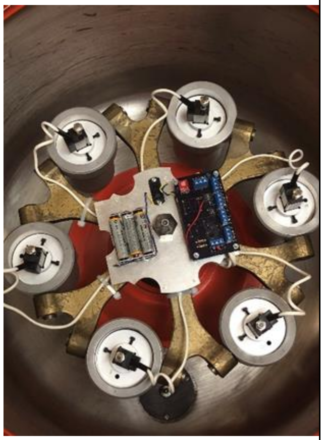
1.	*Weight (syringe+ water)*	Weigh out and record the mass of 100 mL water in a syringe	
2.	Weight (syringe)	Add the water to the centrifuge samples after the prescribed dry consolidation time, and re-weigh the syringe. The difference in mass corresponds to the volume of water added	

8.8. Resuming the centrifuge:

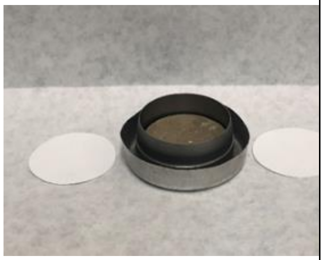
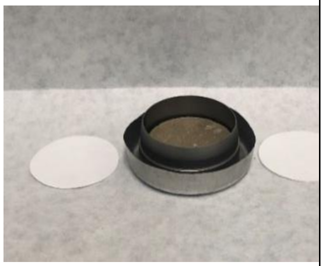
1.	*Restarting the Centrifuge*	After addition of the water, the centrifuge should be re-started to bring the load on the samples to the appropriate level during the swelling phase of the test	
2.	*Stopping the Test*	After reaching secondary swelling, the centrifuge should be turned off, and the samples allowed to rebound for a minimum of 1 hour, while still collecting height data.	

8.9. Test Breakdown:

1.	Total mass (cup+c.c)	After removing the centrifuge buckets from the rotor, weigh and record the mass of each centrifuge bucket and cup. This value will be used in the calculation of water lost during the test.	
2.	Mass (cup+water)	Weigh and record the mass of the permeameter cup. This value will be used in the calculation of the height of ponded water on the sample during the test, which will affect the applied overburden load from the LVDT probe shaft.	
3.	Water ponded? (Yes or No)	If water is still ponded over the soil specimen, then the sample is expected not to have undergone any drying shrinkage during the test. If water is not ponded, the sample may have experienced drying shrinkage, and the test may give erroneous results and should be discarded.	
4.	Water in CC? (Yes or No)	This check is performed to assess whether the permeameter cup is leaking or not during the test.	

5.	Water in Centrifuge? (Yes or No)	This check is performed to ensure that no centrifuge bucket leaks during the test.	

8.10. Measurement of the final water content:

1.	Container Number	Record the name of the drying container used to hold each sample.	
2.	Weight of Tray	Measure and record the mass of teach container.	
3.	Weight of ws+t (with fps)	Remove the specimen from the permeameter cup, and remove the porous disks from the faces of the specimen. Blot the surfaces of the cutting ring and the surfaces of the sample, to remove any extraneous moisture. Measure and record the wet mass of the sample with filter papers.	
4.	Weight of ds+t (with fps)	After a minimum of 16 hours of drying in an oven, measure the mass of the dry sample. The final water content calculated after the test will be used to back-calculate the actual initial conditions used in the test.	

Note 1: Once test has run to completion, the centrifuge should be stopped, and the samples allowed to rebound for a minimum of 1 hour, so as to minimize the change in water content during the subsequent step

Note 2: After allowing partial rebound the onboard data acquisition should be turned off and the centrifuge buckets should be removed immediately. It is important that the final steps of the test procedure be performed quickly to minimize water intake by the samples between the time when the DAQ is turned off and the samples are finally removed from contact with the ponded water.

Note 3: The mass of the total apparatus should be measured and recorded, then the permeameter should be removed and its mass measured, after any excess water has been wiped from the outside.

Note 4: After weighing the permeameter, the testing ring should be removed from the permeameter cup, discarding all remaining ponded water. The top and bottom porous discs should then be removed, and any excess water should be wiped from the ring. The surfaces of the sample should be lightly blotted if any ponded moisture is evident, such that the measured final moisture content is not affected by extraneous water. Moisture content values should be measured and recorded for each sample after swelling. These values will also be used to back-calculate the compaction moisture content for each sample.

9. CALCULATIONS

9.1. The collected centrifuge test data must be analyzed to produce the stress-swell curve needed for PVR prediction using the calculation spreadsheet. All data recorded during preparation should be input into the Centrifuge Test Data Template spreadsheet, and the swell data from the data acquisition system should be uploaded to the spreadsheet. Figure 5 shows a section of the Data Input sheet; the swell data is added on the left-hand side, and time values for determining the seating height, initial height, and start of swell are input in the pink shaded boxes on the right-hand side. The seating height is taken as the sample height during the seating load of 3-5 G's, and the initial height is taken as the height near the end of the compression stage when sample heights have stabilized.

dT (hr)	dS1 (cm)	dS2 (cm)	dS3 (cm)	dS4 (cm)	dS5 (cm)	dS6 (cm)	g-level	Adjusted Time	Row Number
								-1.74	2
								-1.72	3
								-1.70	4
								-1.68	5
								-1.66	6
								-1.64	7
								-1.62	8
								-1.60	9
								-1.58	10
								-1.57	11
								-1.55	12
								-1.53	13
								-1.51	14
								-1.49	15
								-1.47	16
								-1.45	17
								-1.43	18
								-1.41	19
								-1.39	20

Instructions for Inputting Data	
1	Import Data from processed text file
2	Start import at Row 2
3	Select "Comma" Delimited
4	Leave cell formatting "General"
5	Before Inserting Data, Click on "Properties"
6	Deselect "Save Query Definition"
7	Select "Overwrite Existing Cells, delete Unused Cells"
8	Import Data to Cell A2, and fill in Cells for 'Adjusted Time' and 'Row Number'
9	Enter the Row Number corresponding to the Seating Load, Initial, and Begin of Swell
10	Use Graphs for reference when selecting row numbers

	Enter Row #	Time (hr)	S1 (cm)	S2 (cm)	S3 (cm)	S4 (cm)	S5 (cm)	S6 (cm)
Seating:	3.00	0.02	-0.01	0.00	-0.01	-0.01	-0.01	-0.01
Initial:	92.00	1.74	-0.01	-0.01	-0.01	-0.02	-0.01	-0.01
Begin of Swell:	92.00	1.74						

Target G - Level:	20
Actual G - Level:	22.11

Values to Plot for Visual Checks			
9.94	-1.72	-0.02	0.00
-0.28	-1.72	0.00	0.00

Figure 5: Data input sheet in centrifuge test data template.

9.2. Figure 6 shows the initial compression data for each sample and the point of measurement for seating and initial heights. Figure 7 shows the swelling curves for each sample after the water has been added and the swell test has begun. It can be seen that the samples reach the end of primary swell within approximately 4 hours, but the samples are allowed to swell for a total of 16 hours to ensure that they are well into the secondary swell stage.

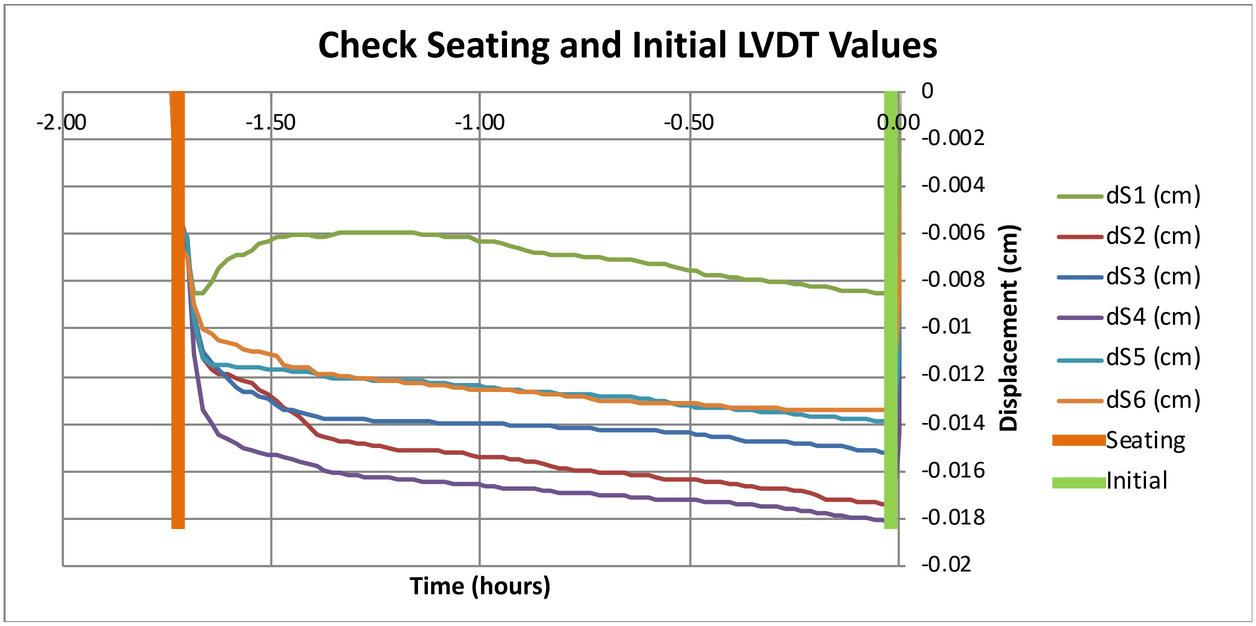


Figure 6: Determining seating and initial height values.

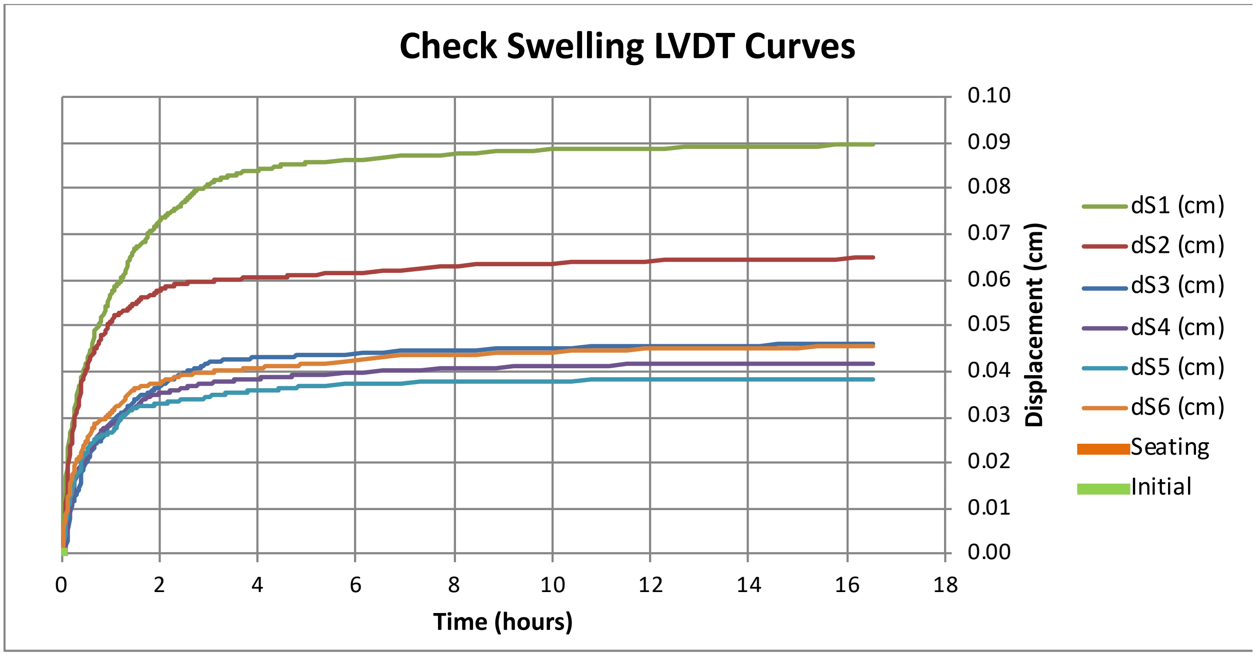


Figure 7: Sample time-swell curves from centrifuge data.

9.3. Figure 8 shows the swell-time curve for a sample in semi log space, along with representative points for both primary and secondary swell. The point representing the end of primary swell is shown in yellow in Table 2 and is determined as the intersection of the lines created by the red points and the green points in Figure 8. These points are determined by adjusting the time values in yellow in Table 2 such that the first two points are within the area of primary swell, and the second two points are within the area of secondary swell.

Table 2: Determination of the slope of primary and secondary swell curves

	Sample Height	Swelling	Time	Slope (ln)	Slope(log)
Primary	1.01496704	3.25%	9.28	1.97%	4.53%
	1.065290712	8.36%	125.32		-0.01
Sec.	1.076681296	9.52%	269.27	0.35%	0.80%
	1.08062405	9.92%	853.49		0.08

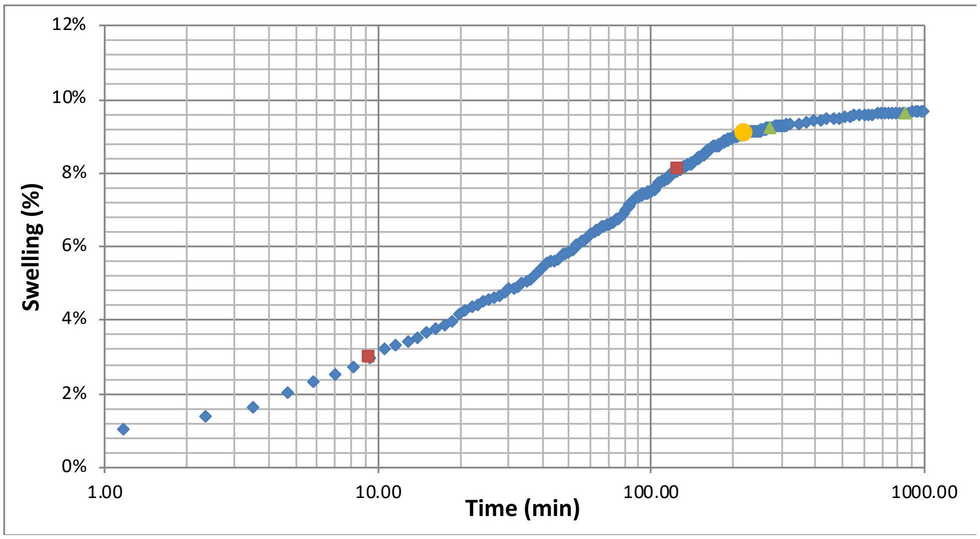


Figure 8: Typical time-swell curve in semi log space, which is useful to determine the end of primary swell.

9.4. Repeating this process with each of the samples allows generation of the information needed to produce a stress-swell curve, as shown in Figure 9. The “swell” values shown in the figure correspond to the swell values at the end of primary swell, as determined using the above method. The “max swell” values shown in the figure are the maximum values determined across the swell-time curve. The stress-swell data shown in Figure 9 can then be used in the PVR Calculation Spreadsheet.

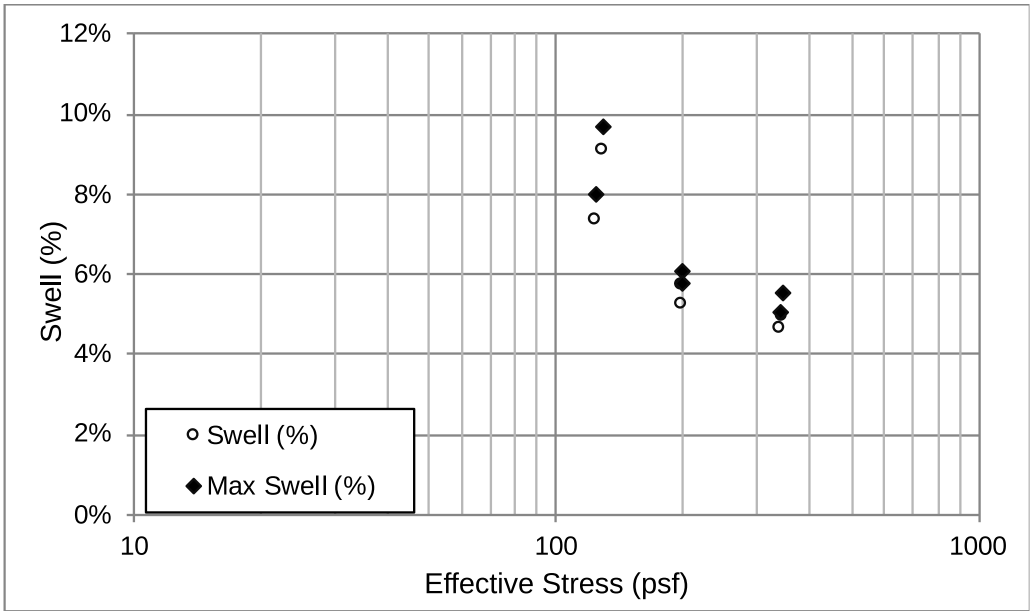


Figure 9: Typical swell-stress curve obtained from a series of Texas swell tests.

10. TEST REPORT

10.1. Report the following information for each sample including the initial dry density, initial moisture content, Lime percentage, dry density and water content after swelling.

10.2. Report the swelling percentage at the applied stress for each sample.

11. EXAMPLE DATASHEET

Project
Operator
Date:
Centrifuge ID:
Target g-level:

		Cup #1	Cup #2	Cup #3	Cup #4	Cup #5	Cup #6	
	Soil:							
	Boring:							
	Depth:							
	Target Conditions							
	Sample Diameter							(in)
	Target Height							(in)
	Target Moisture Content							( )
	Target Dry Density							(pcf)
	Target Moist Mass to add							
	Sample Preparation							
1	Mass (cup):							(g)
2	Mass (ring, grease, fp)							(g)
3	Mass (r+fps+dis+g)							(g)
4	Mass (1 fp, t. disk)							(g)
5	Height (1 fp, t. disk)							(in)
6	Height (1 fp, b.d)							(in)
7								
a	Height @ Center							(in)
b	Height @ 0							(in)
c	Height @ 90							(in)
d	Height @ 180							(in)
e	Height @ 270							(in)
8	Mass (ring+soil)							(g)
9	Weight (cup assembly)							(g)
10	Weight (Centr. Bucket + Cup)							(g)
(11)	Extra Spacer Thickness							(in)
12	Weight (Syring+water)							(g)
13	Weight (Syringe)							(g)
	After 48 Hours							
15	Total mass cup+c.c							g
16	Mass (cup+water)							g
	Water ponded?							
	Water in CC?							
	Water in Centrifuge							
	Oven Drying Sample							
17	Container Number							
18	Weight of tray							g
19	Weight of ws+t							g
20	Weight of ds+t							g

Notes:

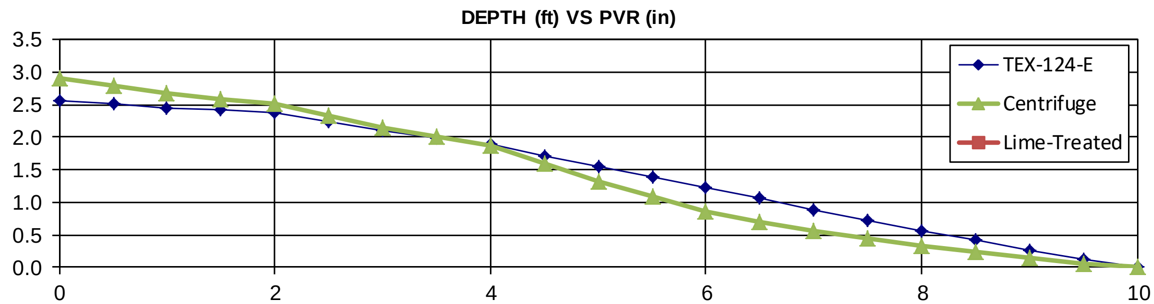
19 a	**Post Test Air-Dry 1**							
	Mass tray, ring, soil, fp							
	Diameter 1							(in)
	Diameter 2							(in)
	Diameter 3							(in)
	Thickness 1							(in)
	Thickness 2							(in)
	Thickness 3							(in)
	Thickness 4							(in)
	Thickness 5							(in)
19 b	**Post Test Air-Dry 2**							
	Mass tray, ring, soil, fp							
	Diameter 1							(in)
	Diameter 2							(in)
	Diameter 3							(in)
	Thickness 1							(in)
	Thickness 2							(in)
	Thickness 3							(in)
	Thickness 4							(in)
	Thickness 5							(in)
19 c	**Post-Test Oven Dry**							
	Mass tray, ring, soil, fp							
	Diameter 1							(in)
	Diameter 2							(in)
	Diameter 3							(in)
	Thickness 1							(in)
	Thickness 2							(in)
	Thickness 3							(in)
	Thickness 4							(in)
	Thickness 5							(in)

Appendix B: PVR Calculations

Borings from FM 2

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	1
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

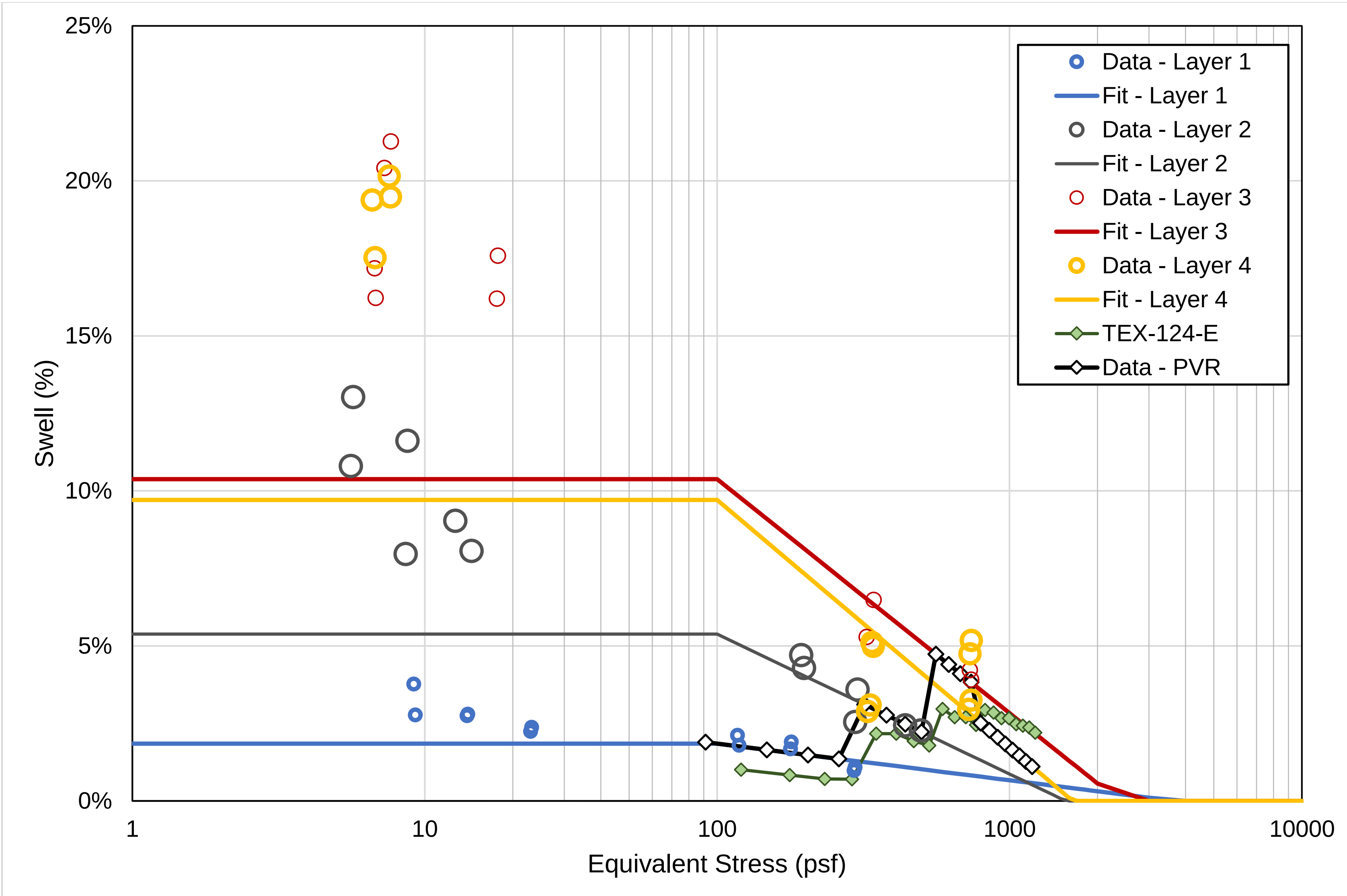
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.89**	**2.56**	
1.0	1	FM2	148	2.78	2.45	
2.0	1	FM2	261	2.59	2.37	
3.0	2	FM2	380	2.32	2.11	
4.0	2	FM2	500	2.01	1.89	
5.0	3	FM2	620	1.59	1.55	
6.0	3	FM2	739	1.08	1.24	
7.0	4	FM2	854	0.70	0.89	
8.0	4	FM2	968	0.44	0.57	
9.0	4	FM2	1082	0.23	0.27	
10.0	4	FM2	1196	0.07	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	54	62	82	92	%
	Plastic Limit	29	41	58	66	%
	Water Content	22%	23%	25%	27%	%
	Dry Unit Weight	92	98	95	90	pcf
	Total Unit Weight	112	121	119	114	pcf
	Thickness	2	2	2	4	ft
	A - Fitting Parameter	-0.012	-0.045	-0.075	-0.080	-
	B - Fitting Parameter	0.0421	0.1441	0.2547	0.2567	-

Texas Department of Transportation

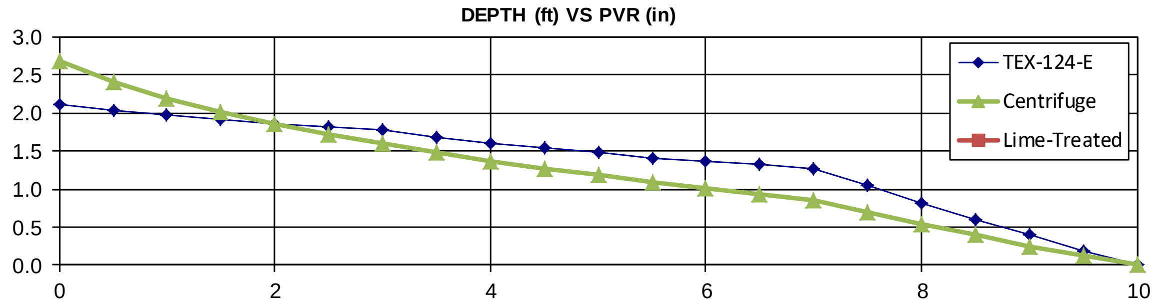


25%
20%
15%
10%
5%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Grimes
Highway:	FM2
CSJ Number:	
District:	Bryan
Boring Number:	2
Date Sampled:	8/13/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

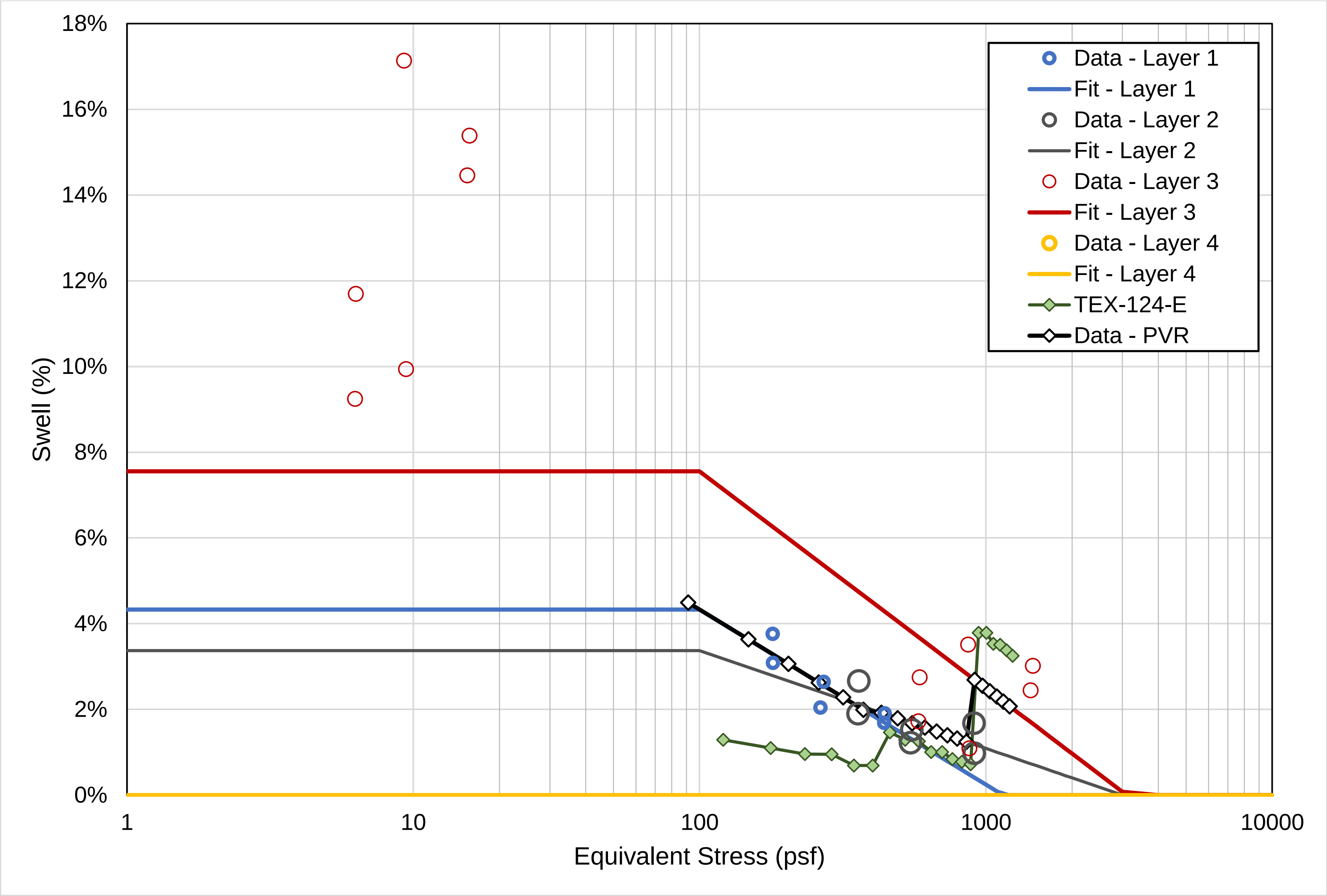
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.68**	**2.12**	
1.0	1	FM2	148	2.41	1.97	
2.0	1	FM2	261	2.01	1.86	
3.0	1	FM2	373	1.72	1.78	
4.0	2	FM2	492	1.48	1.61	
5.0	2	FM2	613	1.27	1.47	
6.0	2	FM2	734	1.09	1.36	
7.0	2	FM2	854	0.93	1.27	
8.0	3	FM2	974	0.69	0.82	
9.0	3	FM2	1093	0.39	0.40	
10.0	3	FM2	1211	0.12	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	FM2	FM2	FM2	FM2	-
Liquid Limit	50	56	100	92	%
Plastic Limit	31	41	79	66	%
Water Content	22%	23%	25%	27%	%
Dry Unit Weight	92	98	95	90	pcf
Total Unit Weight	112	121	119	114	pcf
Thickness	3	4	3	0	ft
A - Fitting Parameter	-0.041	-0.023	-0.051	#DIV/0!	-
B - Fitting Parameter	0.1250	0.0794	0.1768	#DIV/0!	-

Texas Department of Transportation

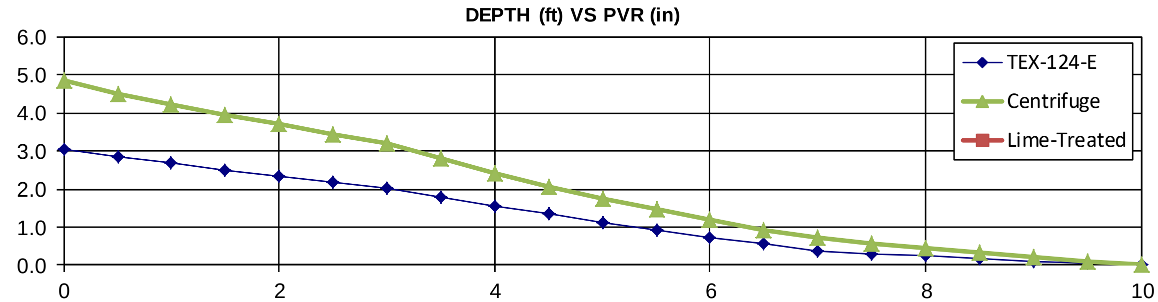


18%
16%
14%
12%
10%
8%
6%
4%
2%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Grimes
Highway:	FM2
CSJ Number:	
District:	Bryan
Boring Number:	3
Date Sampled:	8/13/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

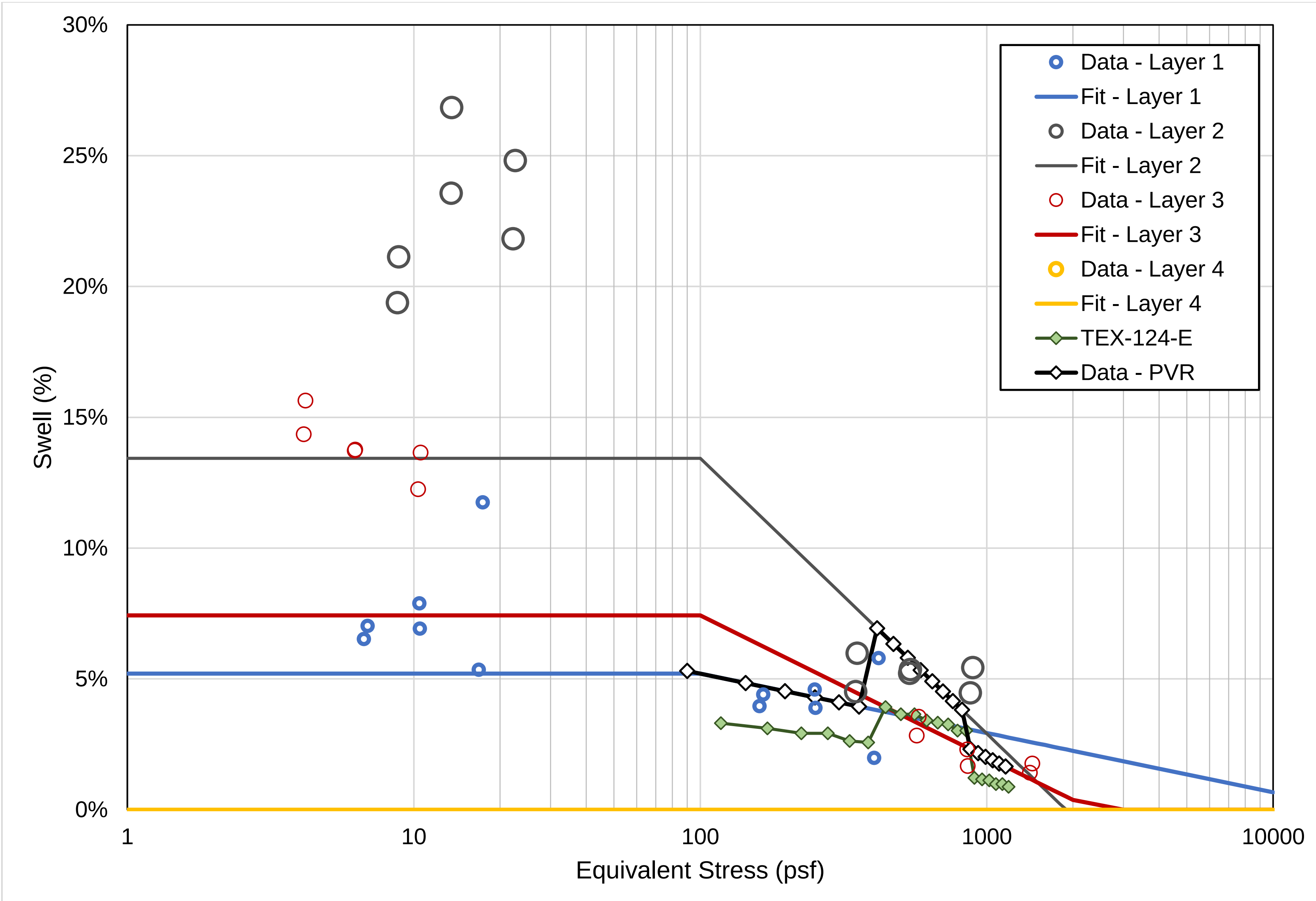
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**4.84**	**3.06**	
1.0	1	FM2	144	4.52	2.68	
2.0	1	FM2	252	3.96	2.33	
3.0	1	FM2	359	3.45	2.02	
4.0	2	FM2	472	2.80	1.56	
5.0	2	FM2	588	2.07	1.14	
6.0	2	FM2	704	1.46	0.74	
7.0	2	FM2	820	0.94	0.38	
8.0	3	FM2	934	0.57	0.24	
9.0	3	FM2	1049	0.32	0.11	
10.0	3	FM2	1163	0.10	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	-	Unit
Soil:	FM2	FM2	FM2	-	-
Liquid Limit	68	88	94	-	%
Plastic Limit	54	66	45	-	%
Water Content	22%	30%	29%	-	%
Dry Unit Weight	88	89	89	-	pcf
Total Unit Weight	107	116	114	-	pcf
Thickness	3	4	3	-	ft
A - Fitting Parameter	-0.023	-0.105	-0.054	-	-
B - Fitting Parameter	0.0974	0.3447	0.1827	-	-

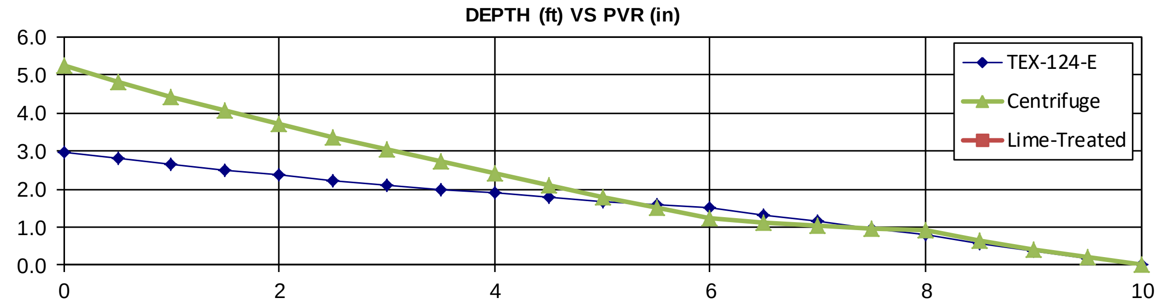
Texas Department of Transportation


30%
25%
20%
15%
10%
5%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Grimes
Highway:	FM2
CSJ Number:	
District:	Bryan
Boring Number:	4
Date Sampled:	8/13/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

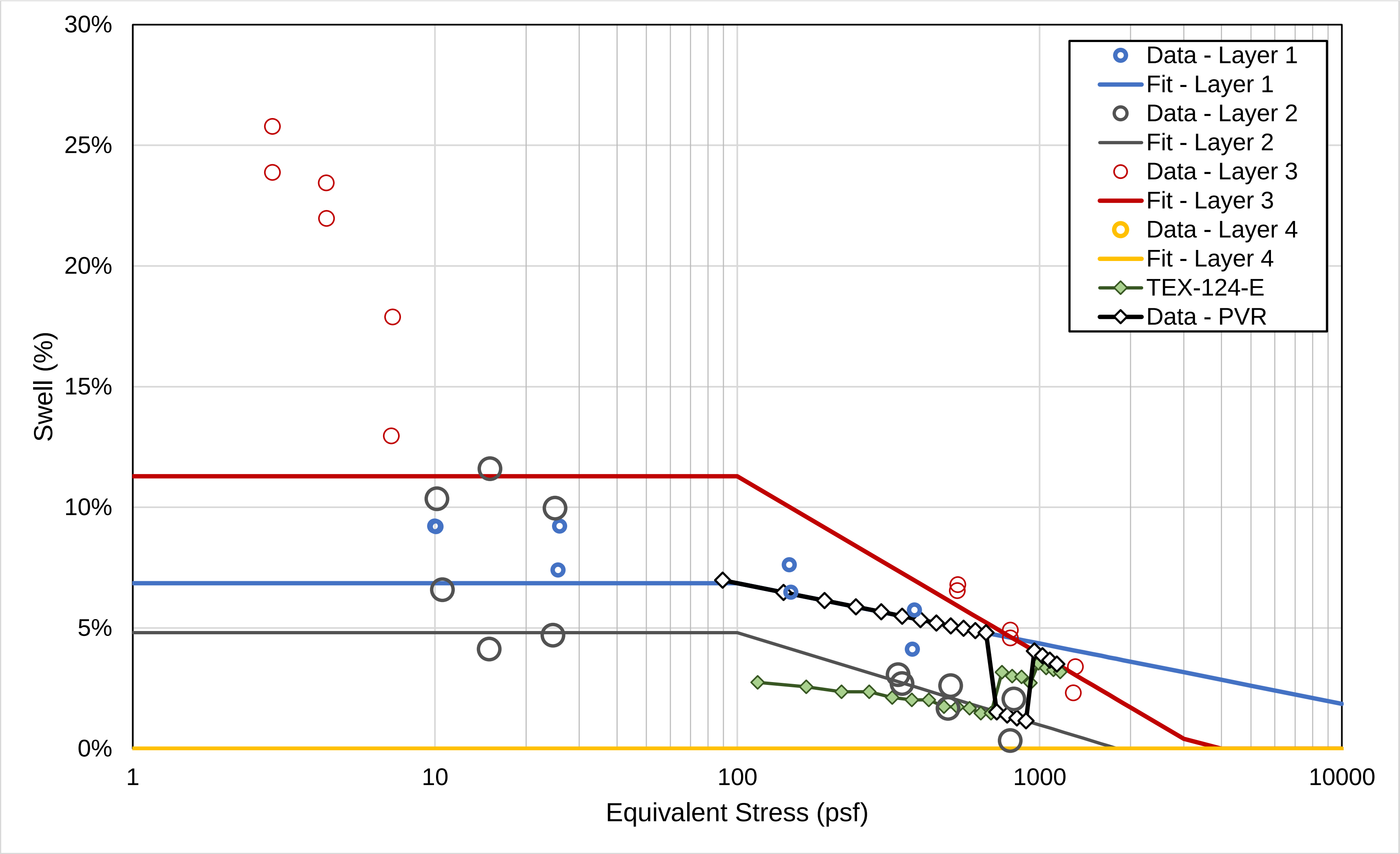
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**5.24**	**2.97**	
1.0	1	FM2	142	4.82	2.65	
2.0	1	FM2	247	4.07	2.37	
3.0	1	FM2	351	3.37	2.12	
4.0	1	FM2	456	2.72	1.89	
5.0	1	FM2	560	2.11	1.69	
6.0	1	FM2	665	1.51	1.51	
7.0	2	FM2	781	1.13	1.14	
8.0	2	FM2	902	0.97	0.80	
9.0	3	FM2	1021	0.66	0.39	
10.0	3	FM2	1140	0.21	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	-	Unit
Soil:	FM2	FM2	FM2	-	-
Liquid Limit	71	87	98	-	%
Plastic Limit	47	63	75	-	%
Water Content	20%	25%	26%	-	%
Dry Unit Weight	87	96	94	-	pcf
Total Unit Weight	104	121	119	-	pcf
Thickness	6	2	2	-	ft
A - Fitting Parameter	-0.025	-0.038	-0.074	-	-
B - Fitting Parameter	0.1185	0.1245	0.2602	-	-

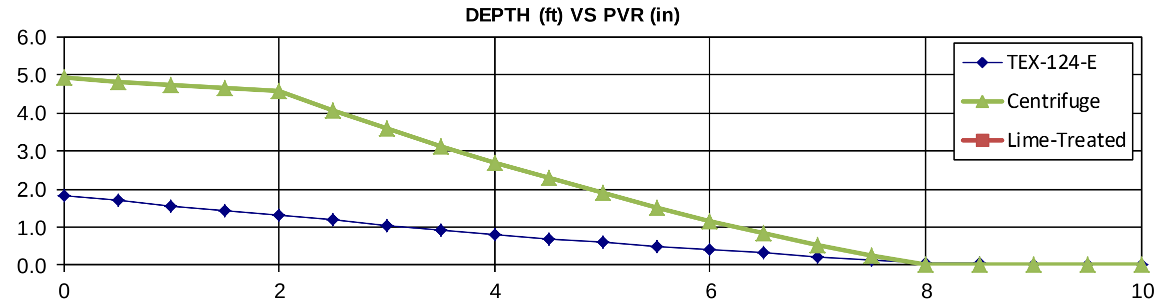
Texas Department of Transportation



30%
25%
20%
15%
10%
5%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	5
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

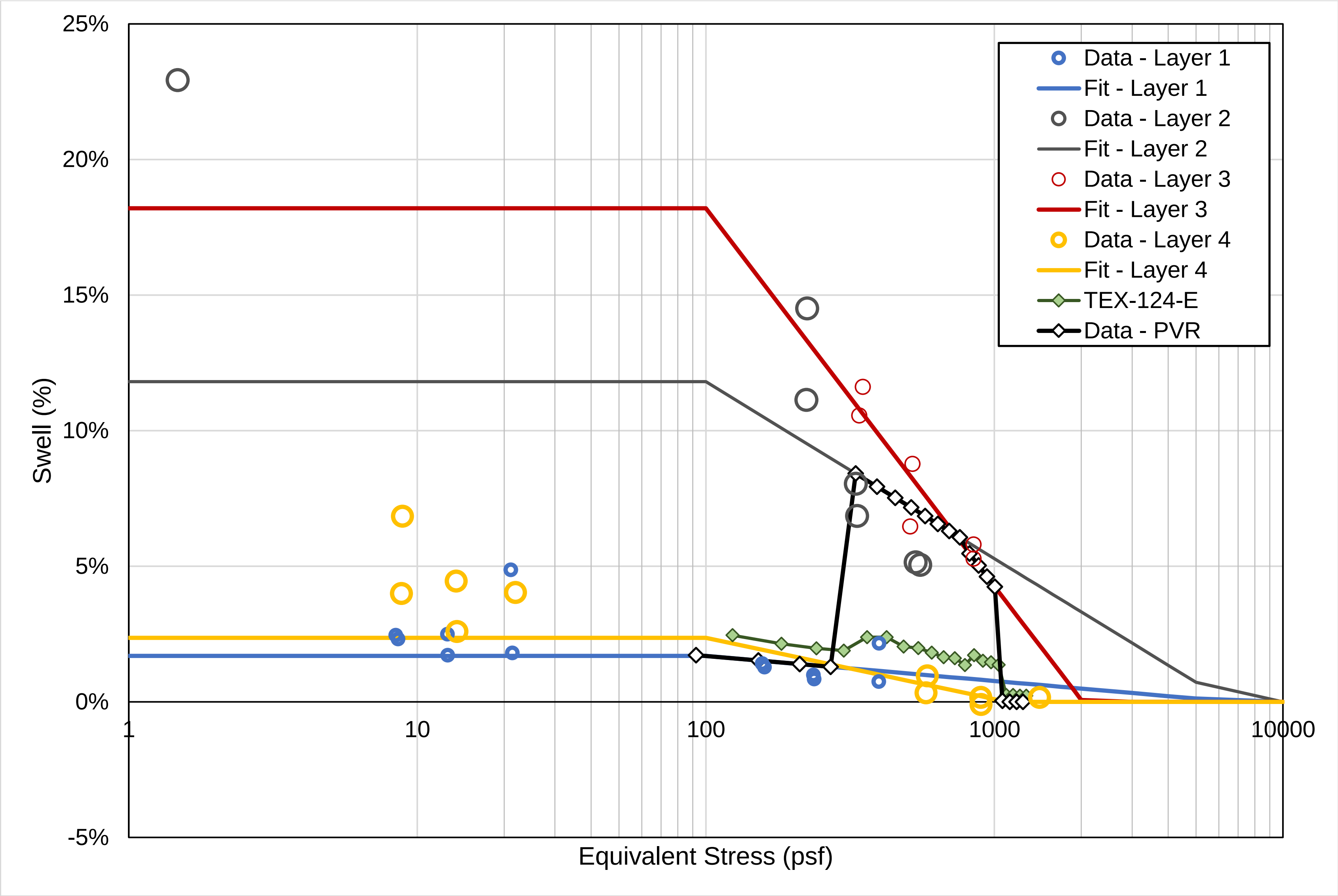
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**4.93**	**1.84**	
1.0	1	FM2	152	4.83	1.57	
2.0	1	FM2	271	4.65	1.33	
3.0	2	FM2	392	4.07	1.05	
4.0	2	FM2	514	3.14	0.81	
5.0	2	FM2	637	2.30	0.60	
6.0	2	FM2	759	1.53	0.42	
7.0	3	FM2	881	0.84	0.23	
8.0	3	FM2	1004	0.26	0.06	
9.0	4	FM2	1130	0.00	0.03	
10.0	4	FM2	1257	0.00	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	62	66	68	40	%
	Plastic Limit	41	42	47	27	%
	Water Content	24%	20%	19%	15%	%
	Dry Unit Weight	95	102	103	110	pcf
	Total Unit Weight	118	122	123	127	pcf
	Thickness	2	4	2	4	ft
	A - Fitting Parameter	-0.009	-0.065	-0.139	-0.023	-
	B - Fitting Parameter	0.0355	0.2486	0.4607	0.0688	-

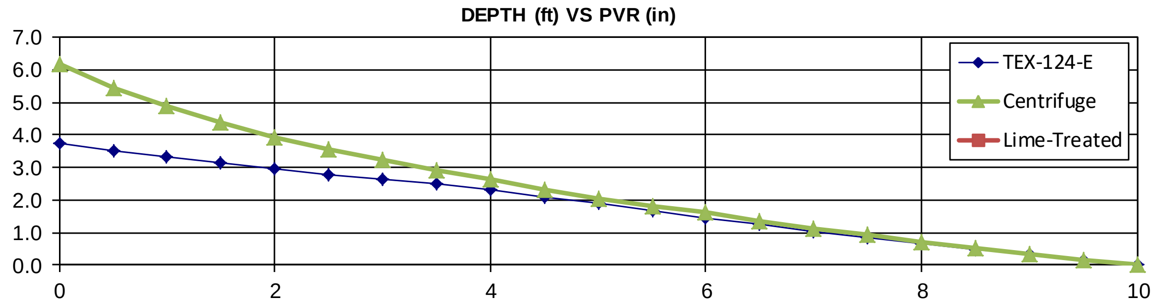
Texas Department of Transportation



25%
20%
15%
10%
5%
0%
-5%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	6
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

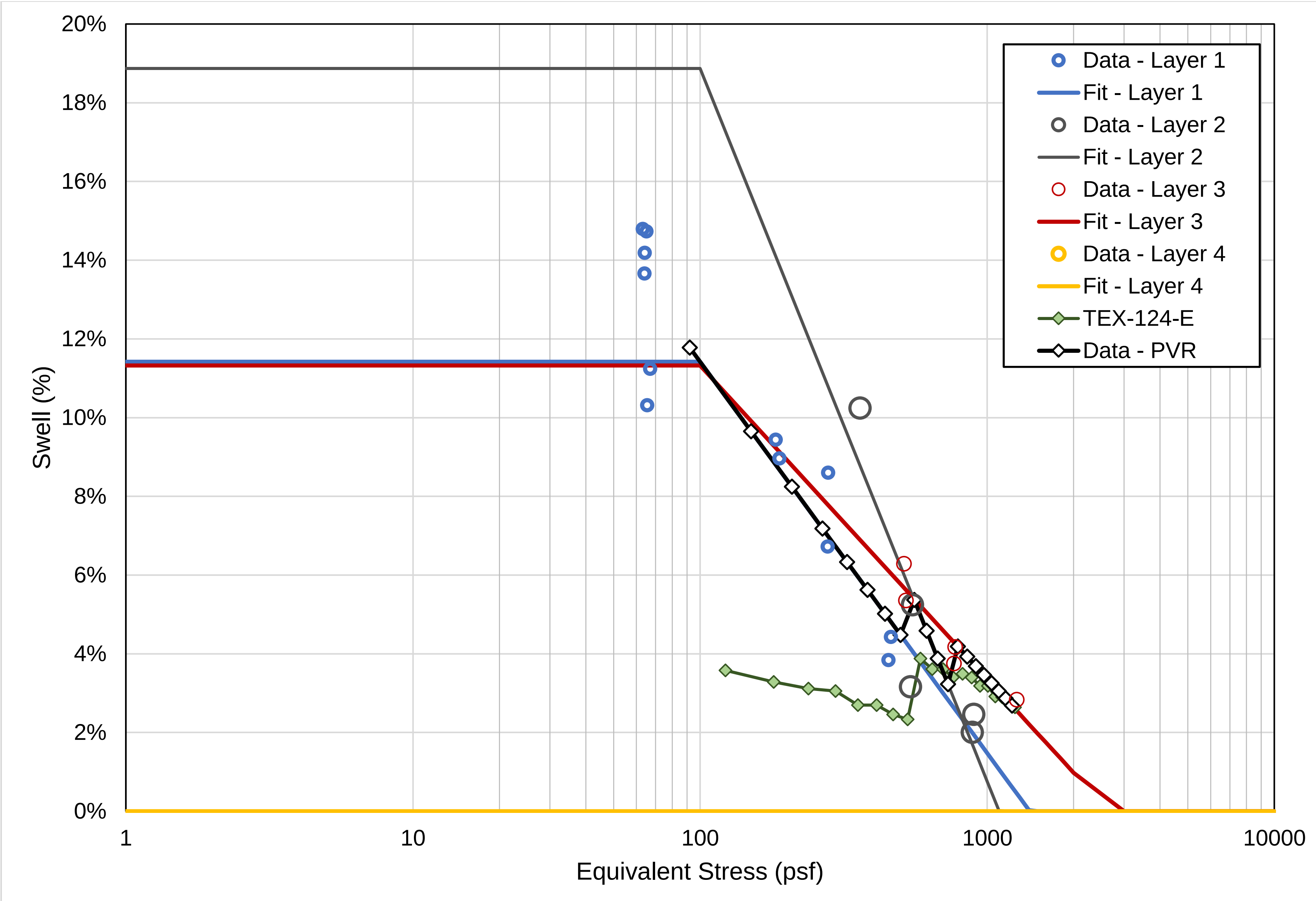
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**6.15**	**3.73**	
1.0	1	FM2	151	5.44	3.32	
2.0	1	FM2	267	4.37	2.95	
3.0	1	FM2	383	3.56	2.63	
4.0	1	FM2	499	2.92	2.34	
5.0	2	FM2	615	2.33	1.89	
6.0	2	FM2	731	1.82	1.47	
7.0	3	FM2	852	1.38	1.05	
8.0	3	FM2	974	0.92	0.67	
9.0	3	FM2	1097	0.52	0.32	
10.0	3	FM2	1220	0.16	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	73	92	88	0	%
	Plastic Limit	50	70	67	0	%
	Water Content	25%	29%	26%	#DIV/0!	%
	Dry Unit Weight	93	90	98	#DIV/0!	pcf
	Total Unit Weight	116	116	123	#DIV/0!	pcf
	Thickness	4	2	4	4	ft
	A - Fitting Parameter	-0.099	-0.181	-0.080	#DIV/0!	-
	B - Fitting Parameter	0.3130	0.5509	0.2723	#DIV/0!	-

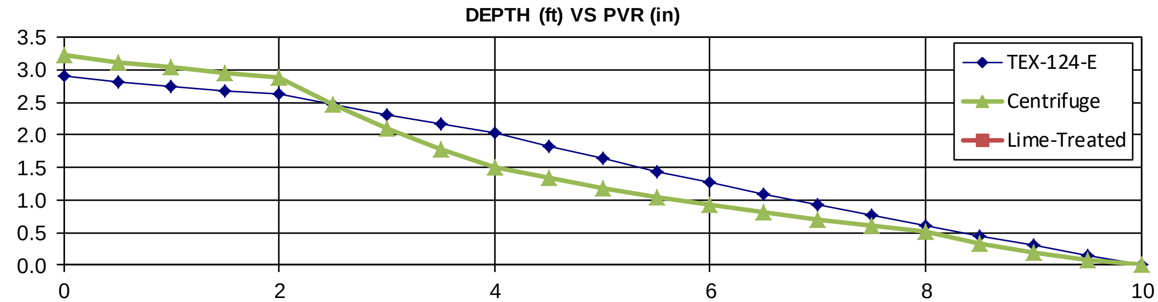
Texas Department of Transportation



20%
18%
16%
14%
12%
10%
8%
6%
4%
2%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	7
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

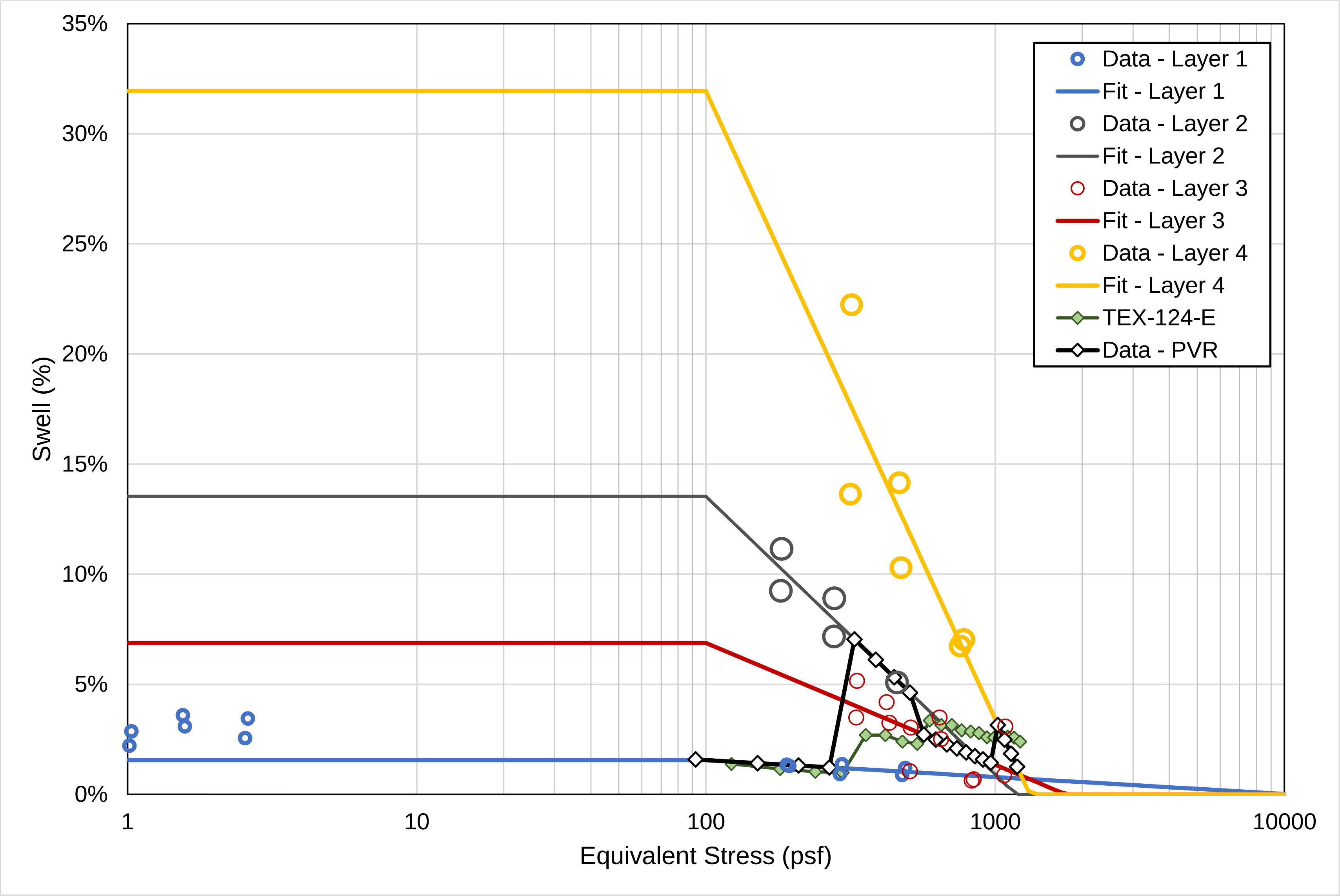
Lime Percentage
0 %

Depth of Treatment
0 ft

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	49	71	88	96	%
	Plastic Limit	31	47	65	70	%
	Water Content	21%	21%	29%	27%	%
	Dry Unit Weight	96	99	88	88	pcf
	Total Unit Weight	116	120	114	113	pcf
	Thickness	2	2	4	2	ft
	A - Fitting Parameter	-0.008	-0.126	-0.055	-0.285	-
	B - Fitting Parameter	0.0309	0.3880	0.1793	0.8901	-

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**3.21**	**2.90**	
1.0	1	FM2	151	3.12	2.74	
2.0	1	FM2	267	2.96	2.62	
3.0	2	FM2	387	2.46	2.30	
4.0	2	FM2	507	1.77	2.02	
5.0	3	FM2	623	1.33	1.63	
6.0	3	FM2	737	1.05	1.27	
7.0	3	FM2	851	0.81	0.93	
8.0	3	FM2	965	0.61	0.62	
9.0	4	FM2	1078	0.33	0.30	
10.0	4	FM2	1191	0.07	0.00	

Texas Department of Transportation

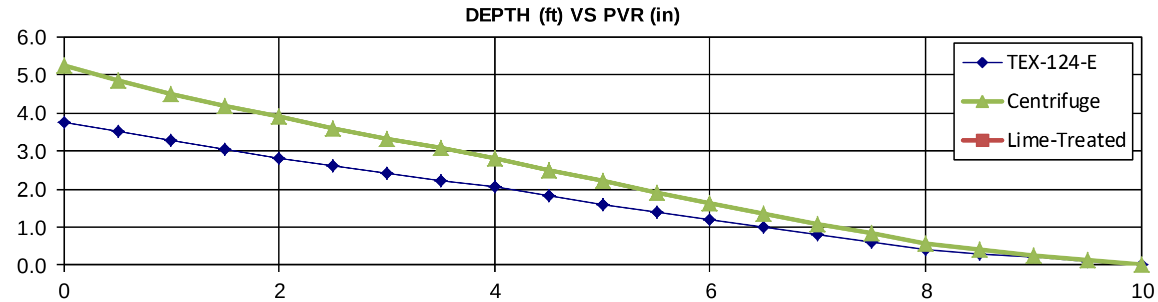


Swell (%)
35%
30%
25%
20%
15%
10%
5%
0%
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Grimes
Highway:	FM2
CSJ Number:	
District:	Bryan
Boring Number:	8
Date Sampled:	8/13/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

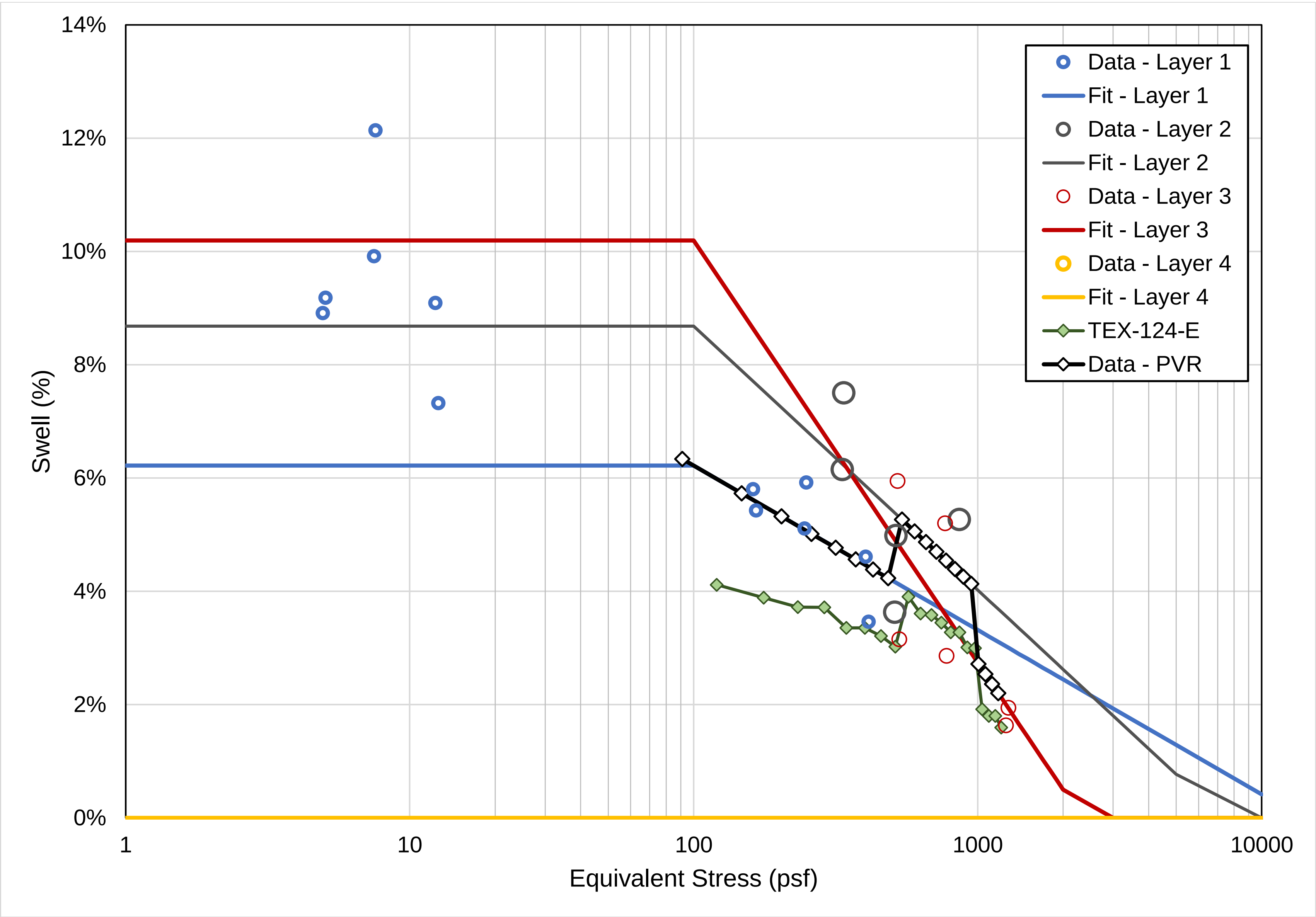
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**5.24**	**3.76**	
1.0	1	FM2	148	4.86	3.28	
2.0	1	FM2	260	4.20	2.83	
3.0	1	FM2	372	3.61	2.43	
4.0	1	FM2	484	3.08	2.05	
5.0	2	FM2	600	2.51	1.60	
6.0	2	FM2	716	1.91	1.18	
7.0	2	FM2	833	1.36	0.79	
8.0	2	FM2	949	0.84	0.43	
9.0	3	FM2	1065	0.43	0.20	
10.0	3	FM2	1181	0.13	0.00	

SOIL Information

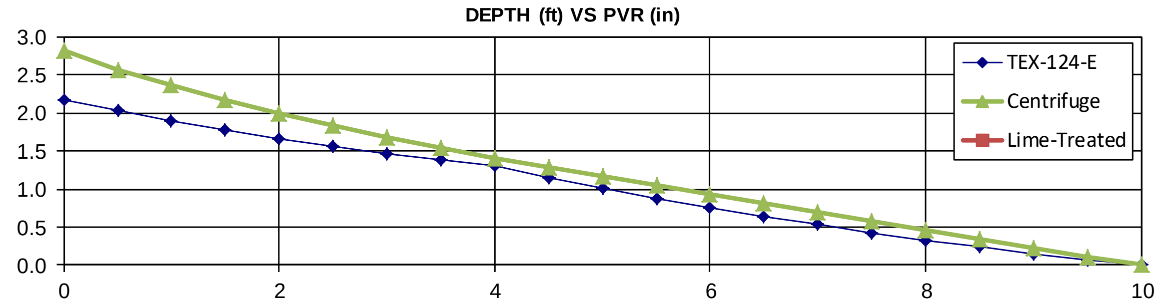
Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	FM2	FM2	FM2	FM2	-
Liquid Limit	83	93	86	0	%
Plastic Limit	61	69	57	0	%
Water Content	22%	27%	27%	#DIV/0!	%
Dry Unit Weight	92	92	91	#DIV/0!	pcf
Total Unit Weight	112	116	116	#DIV/0!	pcf
Thickness	4	4	2	0	ft
A - Fitting Parameter	-0.029	-0.047	-0.075	#DIV/0!	-
B - Fitting Parameter	0.1202	0.1800	0.2510	#DIV/0!	-

Texas Department of Transportation


14%
12%
10%
8%
6%
4%
2%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	9
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

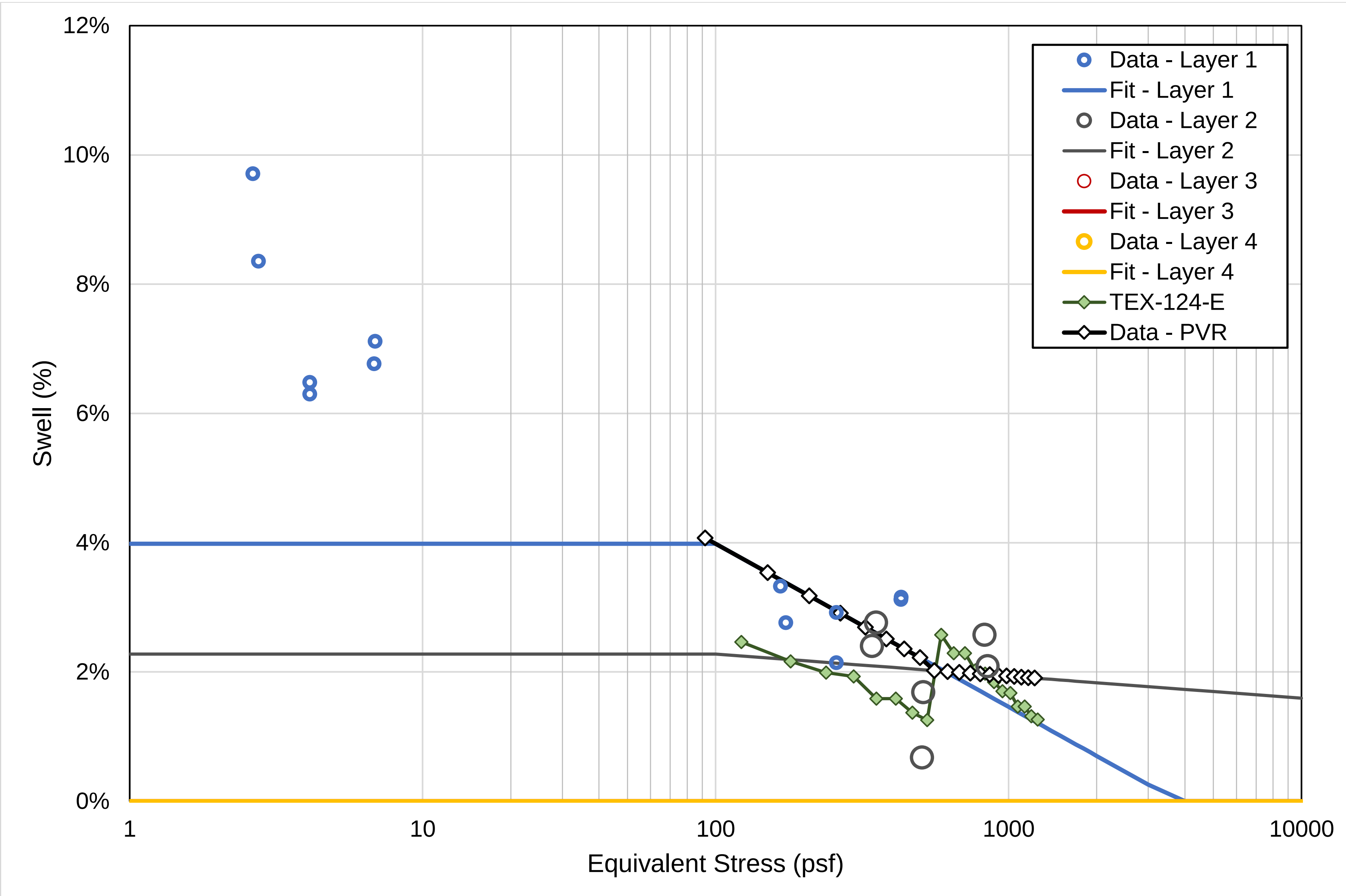
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.82**	**2.17**	
1.0	1	FM2	151	2.57	1.90	
2.0	1	FM2	267	2.17	1.66	
3.0	1	FM2	383	1.83	1.47	
4.0	1	FM2	499	1.54	1.31	
5.0	2	FM2	619	1.29	1.02	
6.0	2	FM2	741	1.05	0.76	
7.0	2	FM2	862	0.81	0.53	
8.0	2	FM2	984	0.58	0.33	
9.0	2	FM2	1106	0.34	0.15	
10.0	2	FM2	1228	0.11	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	61	68	48	0	%
	Plastic Limit	42	51	36	0	%
	Water Content	27%	20%	#DIV/0!	#DIV/0!	%
	Dry Unit Weight	92	102	#DIV/0!	#DIV/0!	pcf
	Total Unit Weight	116	122	#DIV/0!	#DIV/0!	pcf
	Thickness	4	6	2	0	ft
	A - Fitting Parameter	-0.025	-0.003	#DIV/0!	#DIV/0!	-
	B - Fitting Parameter	0.0903	0.0296	#DIV/0!	#DIV/0!	-

Texas Department of Transportation

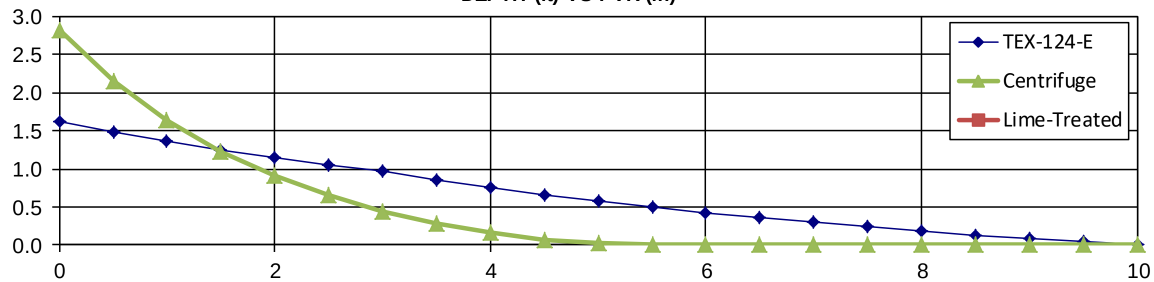


12%
10%
8%
6%
4%
2%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	10
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	

DEPTH (ft) VS PVR (in)



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

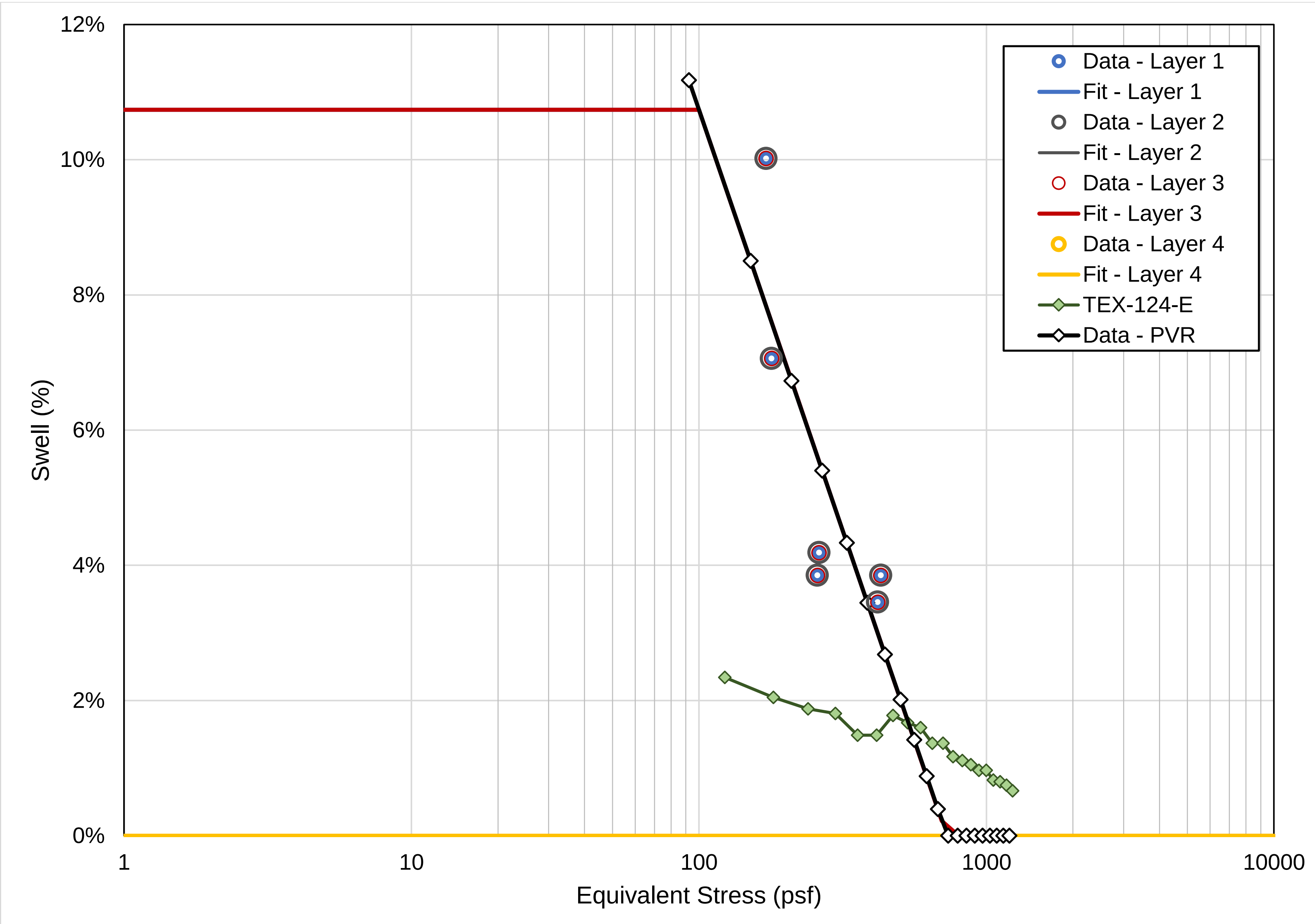
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.82**	**1.63**	
1.0	1	FM2	151	2.15	1.36	
2.0	1	FM2	268	1.23	1.14	
3.0	1	FM2	385	0.65	0.96	
4.0	2	FM2	502	0.28	0.76	
5.0	2	FM2	619	0.08	0.58	
6.0	2	FM2	736	0.00	0.43	
7.0	2	FM2	853	0.00	0.30	
8.0	3	FM2	970	0.00	0.18	
9.0	3	FM2	1087	0.00	0.08	
10.0	3	FM2	1203	0.00	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	52	59	60	0	%
	Plastic Limit	33	40	41	0	%
	Water Content	15%	15%	15%	#DIV/0!	%
	Dry Unit Weight	102	102	102	#DIV/0!	pcf
	Total Unit Weight	117	117	117	#DIV/0!	pcf
	Thickness	3	4	3	0	ft
	A - Fitting Parameter	-0.124	-0.124	-0.124	#DIV/0!	-
	B - Fitting Parameter	0.3564	0.3564	0.3564	#DIV/0!	-

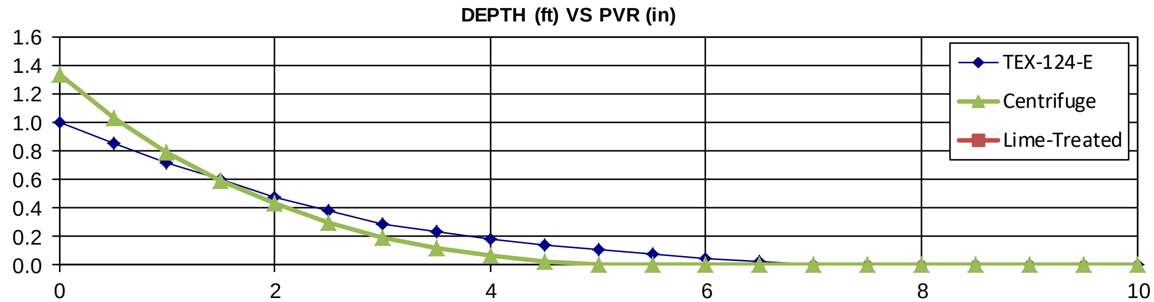
Texas Department of Transportation



12%
10%
8%
6%
4%
2%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	11
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**1.34**	**1.00**	
1.0	1	FM2	146	1.03	0.72	
2.0	1	FM2	256	0.59	0.48	
3.0	1	FM2	366	0.30	0.29	
4.0	2	FM2	483	0.12	0.19	
5.0	2	FM2	603	0.03	0.11	
6.0	2	FM2	722	0.00	0.05	
7.0	2	FM2	842	0.00	0.00	
8.0	3	FM2	953	0.00	0.00	
9.0	3	FM2	1061	0.00	0.00	
10.0	3	FM2	1169	0.00	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	50	38	30	0	%
	Plastic Limit	37	27	13	0	%
	Water Content	17%	12%	14%	#DIV/0!	%
	Dry Unit Weight	94	107	95	#DIV/0!	pcf
	Total Unit Weight	110	119	108	#DIV/0!	pcf
	Thickness	3	4	3	0	ft
	A - Fitting Parameter	-0.055	-0.059	-0.039	#DIV/0!	-
	B - Fitting Parameter	0.1593	0.1679	0.0902	#DIV/0!	-

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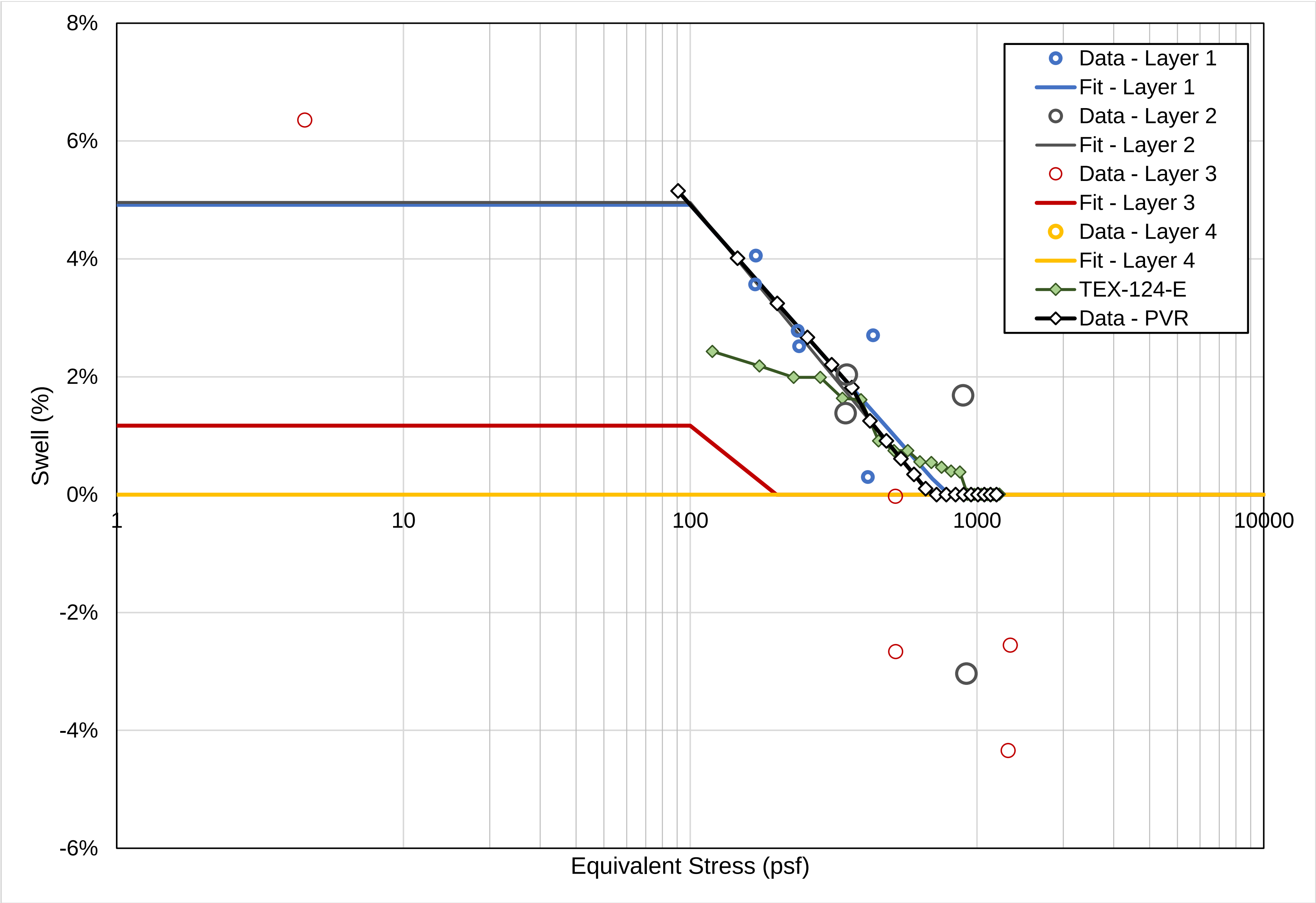
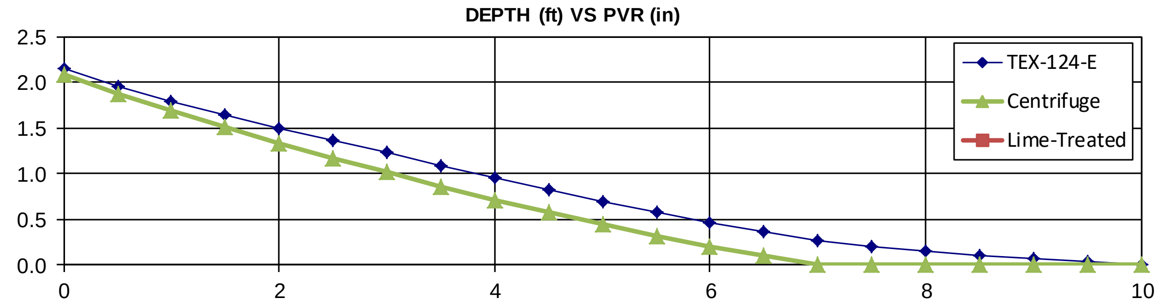


8%
6%
4%
2%
0%
-2%
-4%
-6%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	12
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

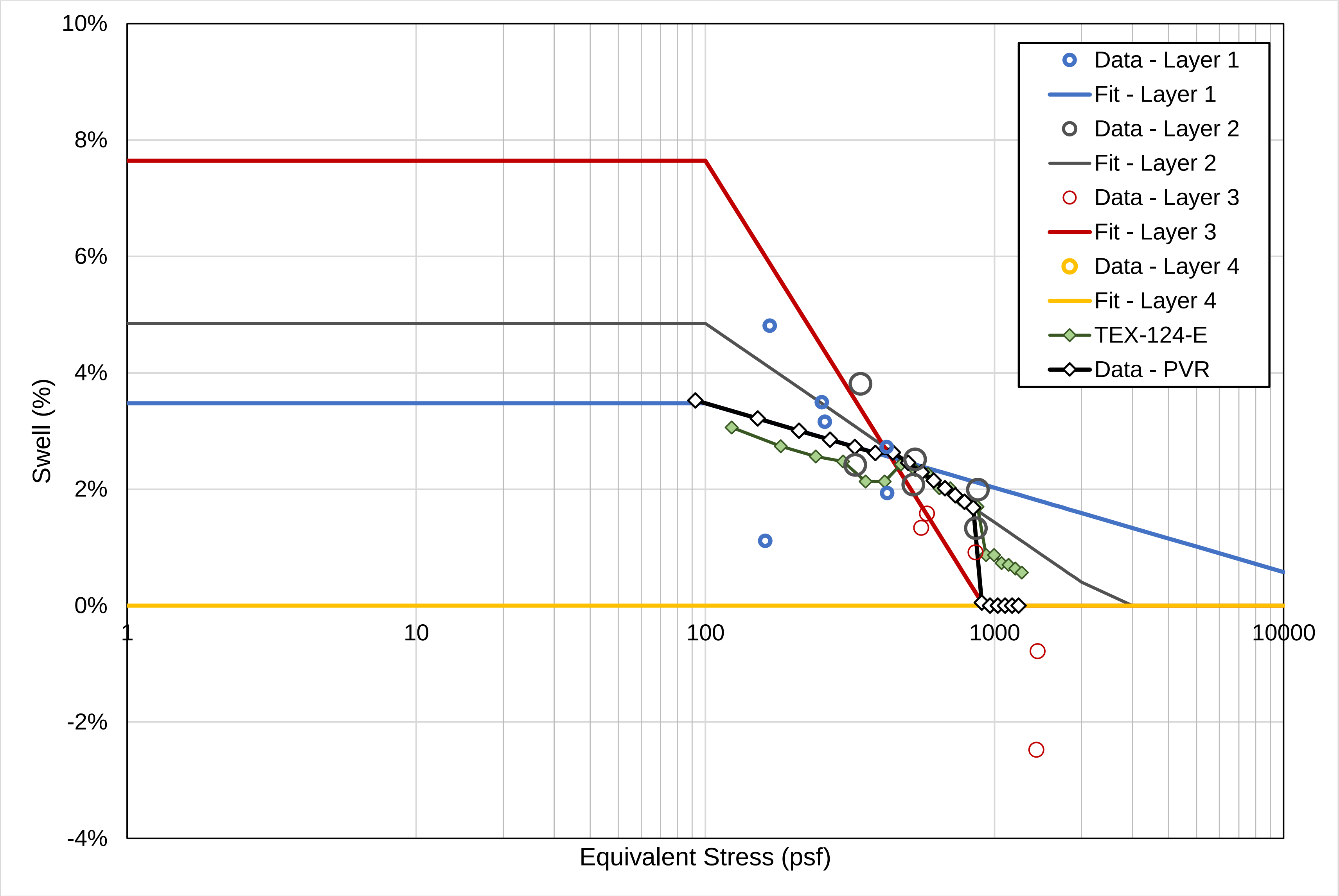
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.09**	**2.15**	
1.0	1	FM2	152	1.88	1.80	
2.0	1	FM2	270	1.51	1.50	
3.0	1	FM2	387	1.18	1.24	
4.0	2	FM2	503	0.86	0.96	
5.0	2	FM2	617	0.57	0.70	
6.0	2	FM2	731	0.32	0.47	
7.0	2	FM2	845	0.10	0.26	
8.0	3	FM2	966	0.00	0.16	
9.0	3	FM2	1089	0.00	0.07	
10.0	3	FM2	1212	0.00	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	56	69	53	0	%
	Plastic Limit	41	51	38	0	%
	Water Content	20%	20%	16%	#DIV/0!	%
	Dry Unit Weight	98	95	106	#DIV/0!	pcf
	Total Unit Weight	118	114	123	#DIV/0!	pcf
	Thickness	3	4	3	0	ft
	A - Fitting Parameter	-0.015	-0.034	-0.079	#DIV/0!	-
	B - Fitting Parameter	0.0638	0.1168	0.2352	#DIV/0!	-

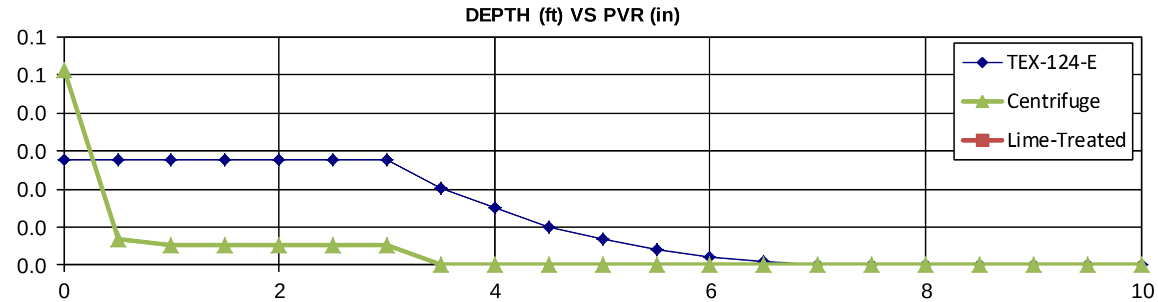
Texas Department of Transportation



10%
8%
6%
4%
2%
0%
-2%
-4%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	13
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

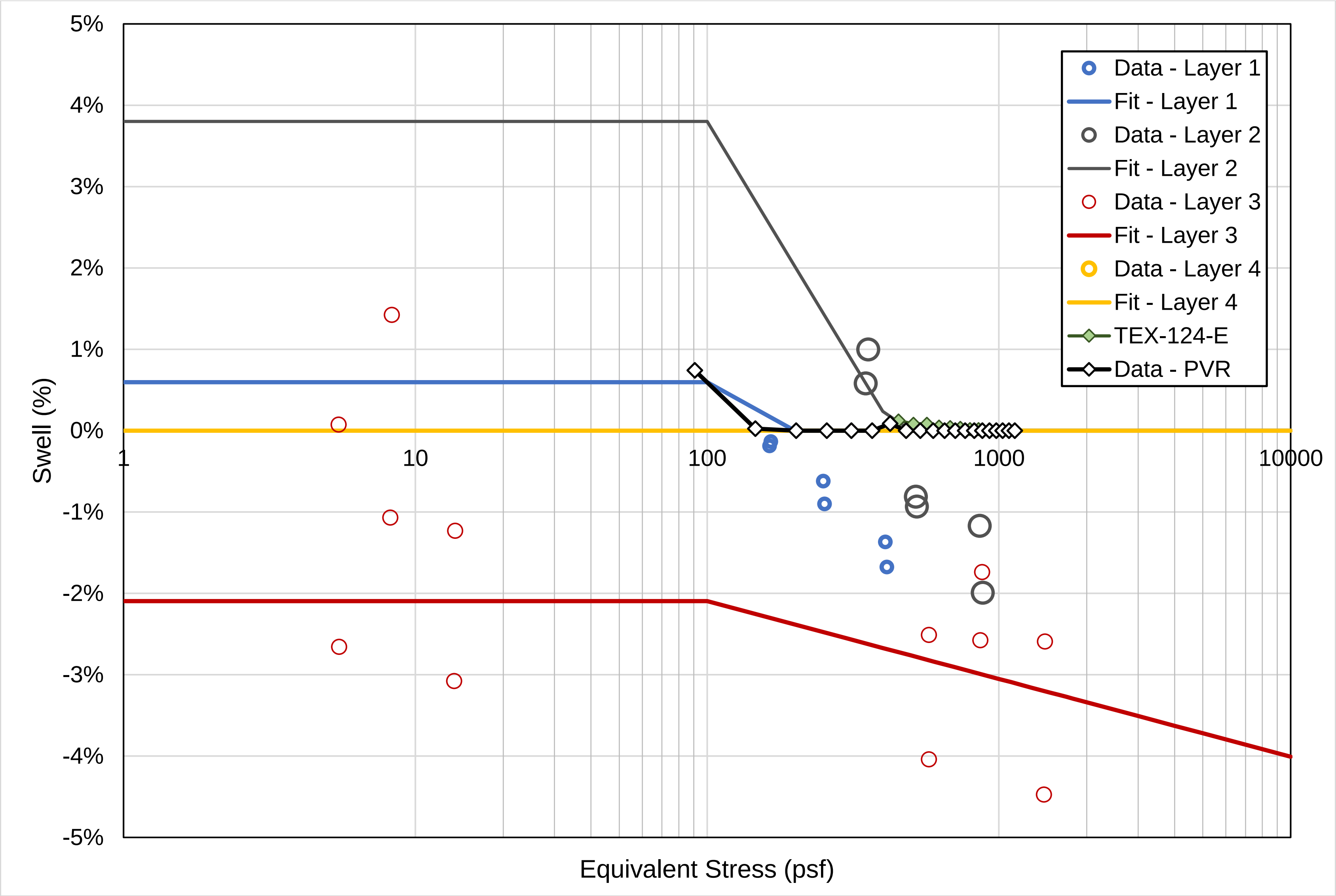
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**0.05**	**0.03**	
1.0	1	FM2	146	0.01	0.03	
2.0	1	FM2	257	0.01	0.03	
3.0	1	FM2	368	0.01	0.03	
4.0	2	FM2	481	0.00	0.02	
5.0	2	FM2	595	0.00	0.01	
6.0	2	FM2	709	0.00	0.00	
7.0	2	FM2	824	0.00	0.00	
8.0	3	FM2	929	0.00	0.00	
9.0	3	FM2	1031	0.00	0.00	
10.0	3	FM2	1133	0.00	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	24	29	22	0	%
	Plastic Limit	8	11	9	0	%
	Water Content	8%	9%	7%	#DIV/0!	%
	Dry Unit Weight	103	105	95	#DIV/0!	pcf
	Total Unit Weight	110	114	102	#DIV/0!	pcf
	Thickness	3	4	3	0	ft
	A - Fitting Parameter	-0.034	-0.059	-0.010	#DIV/0!	-
	B - Fitting Parameter	0.0747	0.1565	-0.0018	#DIV/0!	-

Texas Department of Transportation

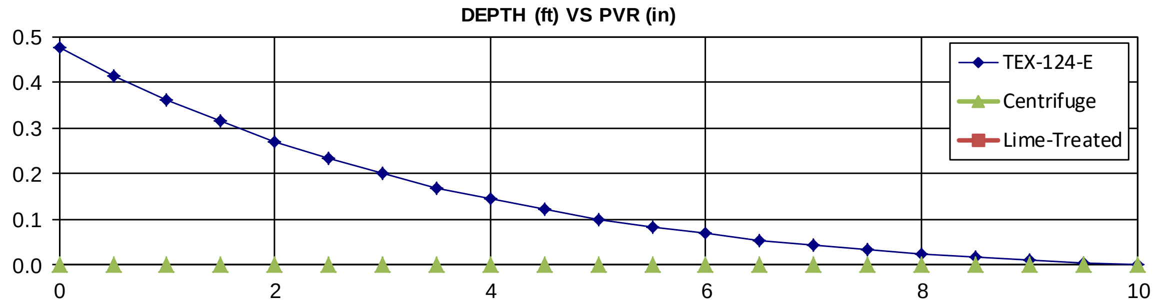


Swell (%)
5%
4%
3%
2%
1%
0%
-1%
-2%
-3%
-4%
-5%
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Grimes
Highway:	FM2
CSJ Number:	
District:	Bryan
Boring Number:	14
Date Sampled:	8/13/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

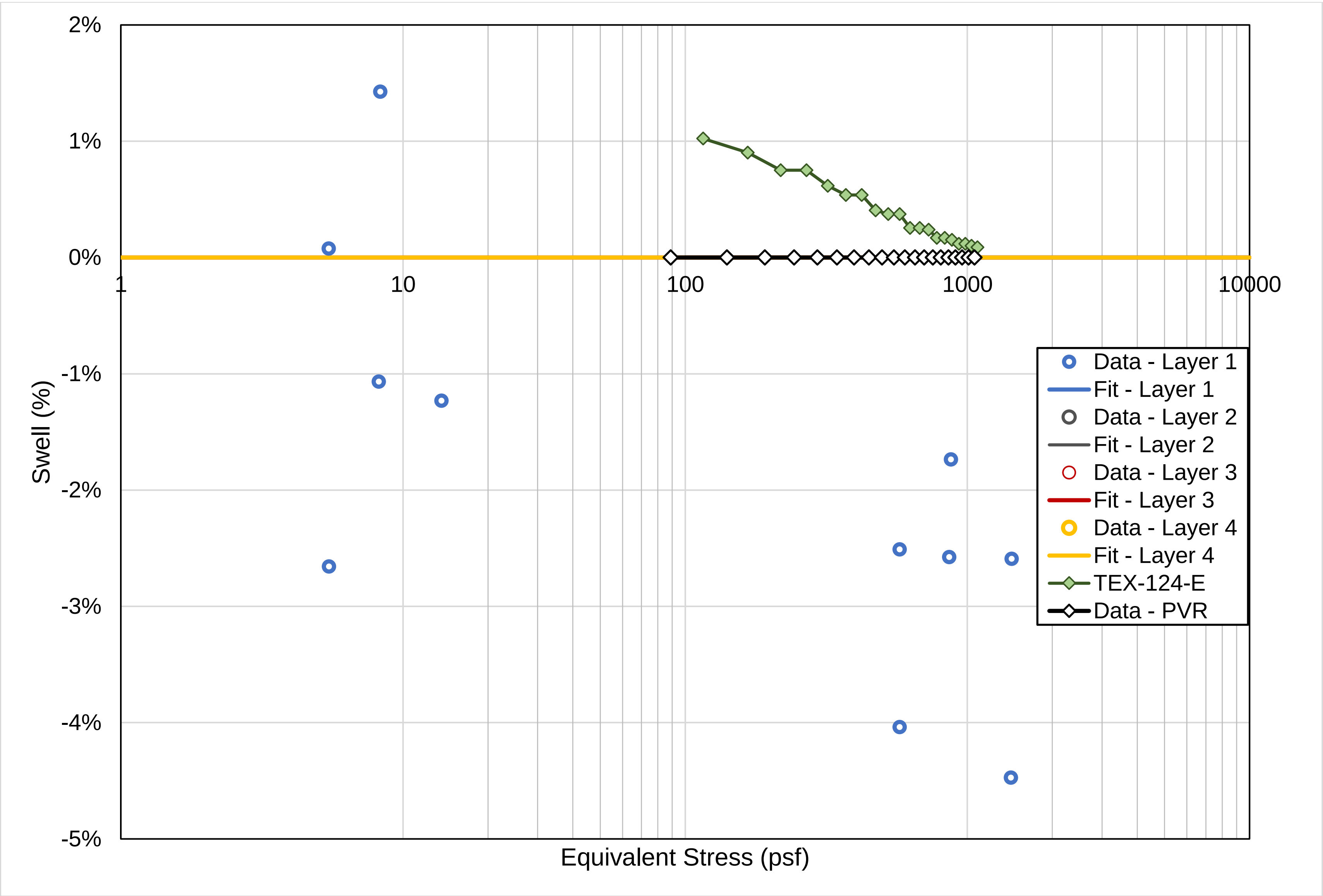
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**0.00**	**0.48**	
1.0	1	FM2	140	0.00	0.36	
2.0	1	FM2	243	0.00	0.27	
3.0	1	FM2	345	0.00	0.20	
4.0	1	FM2	447	0.00	0.14	
5.0	1	FM2	550	0.00	0.10	
6.0	1	FM2	652	0.00	0.07	
7.0	1	FM2	754	0.00	0.04	
8.0	1	FM2	856	0.00	0.03	
9.0	1	FM2	958	0.00	0.01	
10.0	1	FM2	1061	0.00	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	FM2	FM2	FM2	FM2	-
Liquid Limit	39	45	39	0	%
Plastic Limit	24	32	24	0	%
Water Content	7%	#DIV/0!	#DIV/0!	#DIV/0!	%
Dry Unit Weight	95	#DIV/0!	#DIV/0!	#DIV/0!	pcf
Total Unit Weight	102	#DIV/0!	#DIV/0!	#DIV/0!	pcf
Thickness	10	6	2	0	ft
A - Fitting Parameter	-0.008	#DIV/0!	#DIV/0!	#DIV/0!	-
B - Fitting Parameter	-0.0059	#DIV/0!	#DIV/0!	#DIV/0!	-

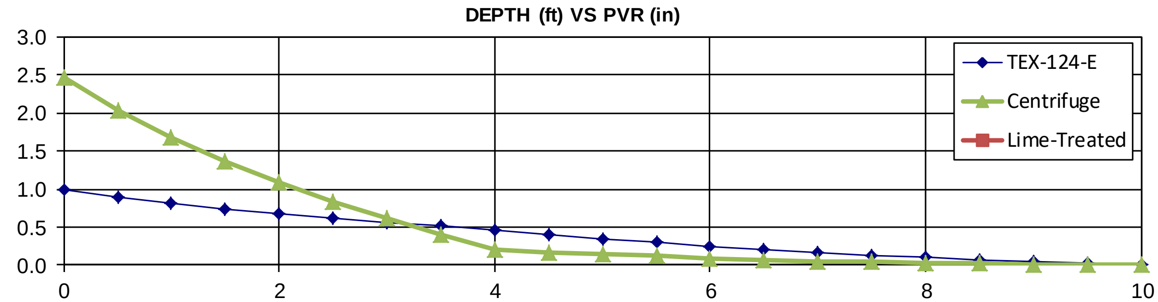
Texas Department of Transportation


2%
1%
0%
-1%
-2%
-3%
-4%
-5%
1
10
100
1000
10000
Swell (%)
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Grimes
Highway:	FM2
CSJ Number:	
District:	Bryan
Boring Number:	15
Date Sampled:	8/13/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

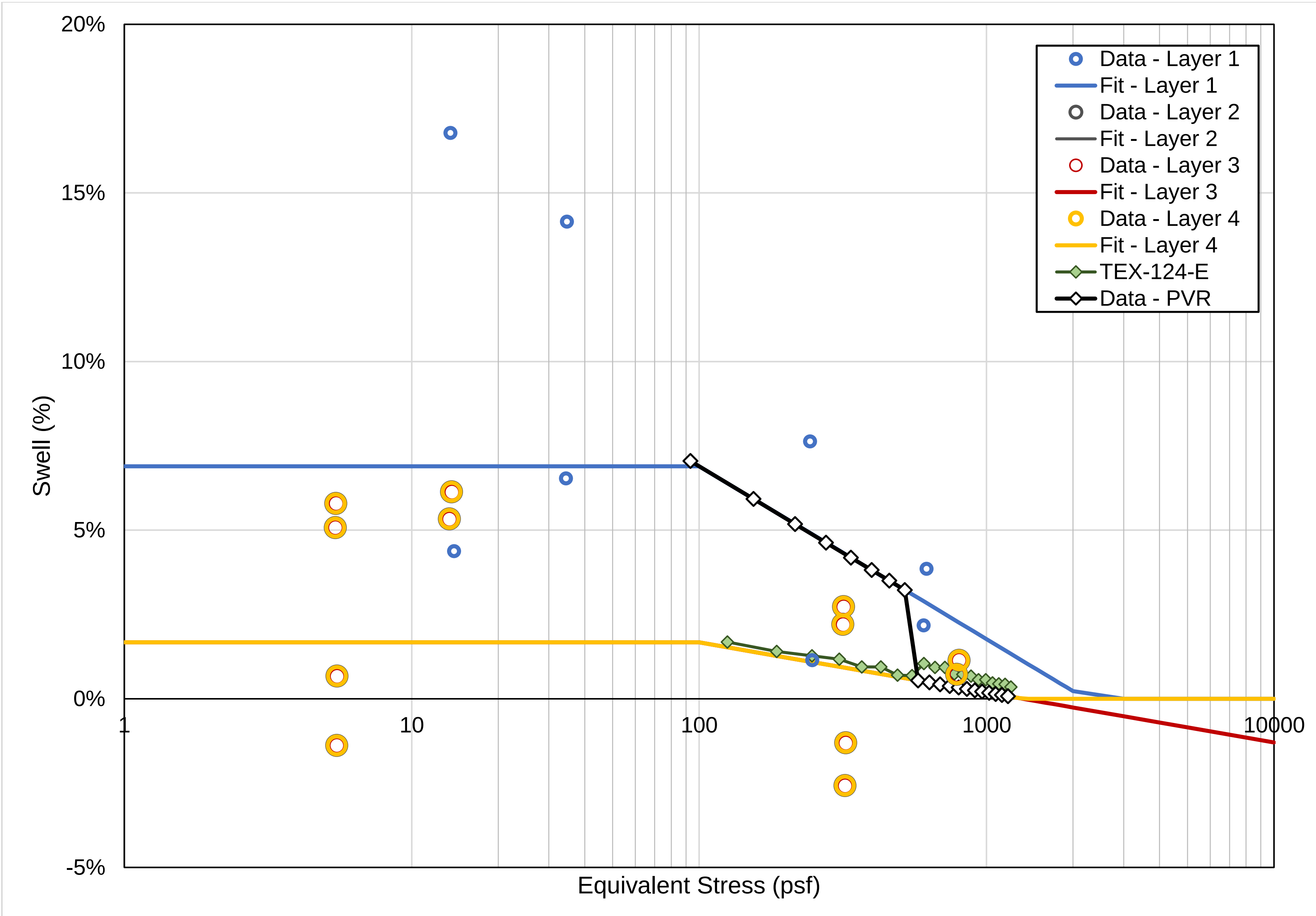
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.46**	**1.00**	
1.0	1	FM2	155	2.04	0.82	
2.0	1	FM2	277	1.37	0.67	
3.0	1	FM2	398	0.84	0.56	
4.0	1	FM2	520	0.40	0.47	
5.0	2	FM2	633	0.17	0.35	
6.0	2	FM2	744	0.12	0.25	
7.0	2	FM2	855	0.08	0.17	
8.0	2	FM2	965	0.04	0.10	
9.0	2	FM2	1076	0.02	0.05	
10.0	2	FM2	1187	0.00	0.00	

SOIL Information

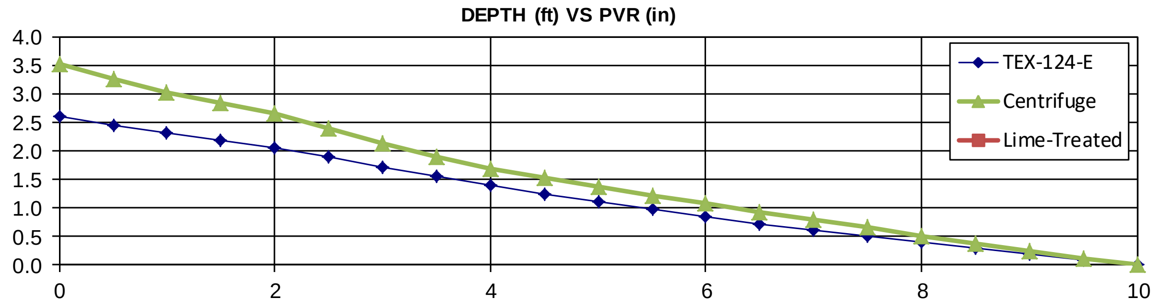
Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	FM2	FM2	FM2	FM2	-
Liquid Limit	41	51	26	42	%
Plastic Limit	26	35	13	25	%
Water Content	17%	14%	14%	14%	%
Dry Unit Weight	104	97	97	97	pcf
Total Unit Weight	121	111	111	111	pcf
Thickness	4	6	0	0	ft
A - Fitting Parameter	-0.051	-0.015	-0.015	-0.015	-
B - Fitting Parameter	0.1715	0.0464	0.0464	0.0464	-

Texas Department of Transportation


Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR
20%
15%
10%
5%
0%
-5%
1
10
100
1000
10000
Swell (%)
Equivalent Stress (psf)

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Grimes
	Highway:	FM2
	CSJ Number:	
	District:	Bryan
	Boring Number:	16
	Date Sampled:	8/13/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

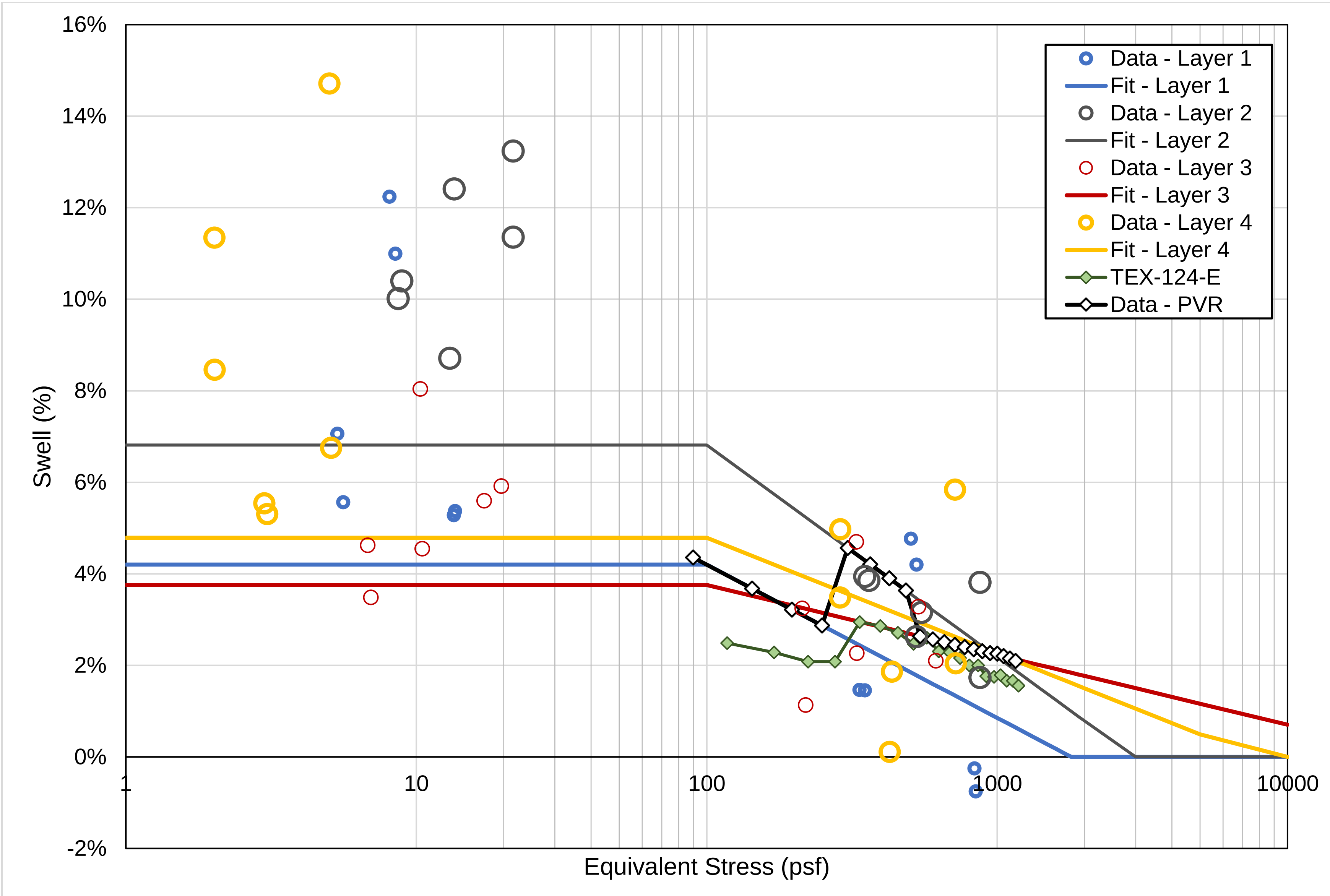
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**3.52**	**2.61**	
1.0	1	FM2	143	3.26	2.32	
2.0	1	FM2	250	2.84	2.07	
3.0	2	FM2	366	2.40	1.72	
4.0	2	FM2	485	1.91	1.41	
5.0	3	FM2	601	1.53	1.12	
6.0	3	FM2	716	1.23	0.85	
7.0	3	FM2	831	0.94	0.61	
8.0	3	FM2	946	0.66	0.40	
9.0	4	FM2	1054	0.39	0.19	
10.0	4	FM2	1158	0.13	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM2	FM2	FM2	FM2	-
	Liquid Limit	59	67	74	83	%
	Plastic Limit	41	50	54	60	%
	Water Content	13%	19%	23%	28%	%
	Dry Unit Weight	94	100	93	82	pcf
	Total Unit Weight	106	119	115	105	pcf
	Thickness	2	2	4	2	ft
	A - Fitting Parameter	-0.033	-0.046	-0.015	-0.025	-
	B - Fitting Parameter	0.1090	0.1608	0.0681	0.0984	-

Texas Department of Transportation

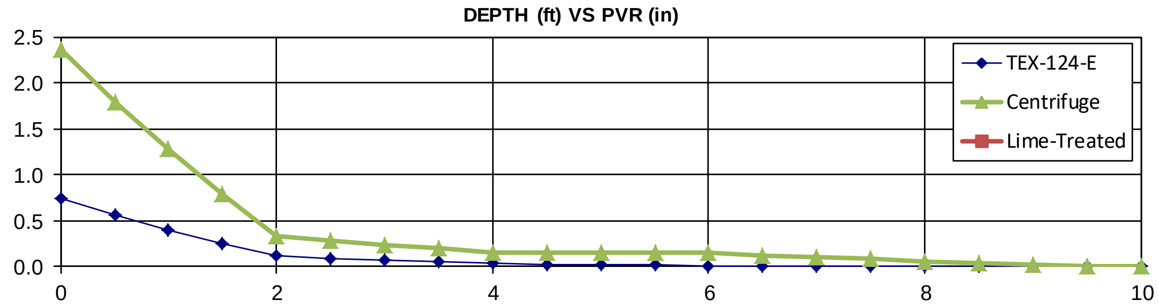


16%
14%
12%
10%
8%
6%
4%
2%
0%
-2%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

Borings from FM 972

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Williamson
	Highway:	FM 972
	CSJ Number:	
	District:	Austin
	Boring Number:	1
	Date Sampled:	8/14/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

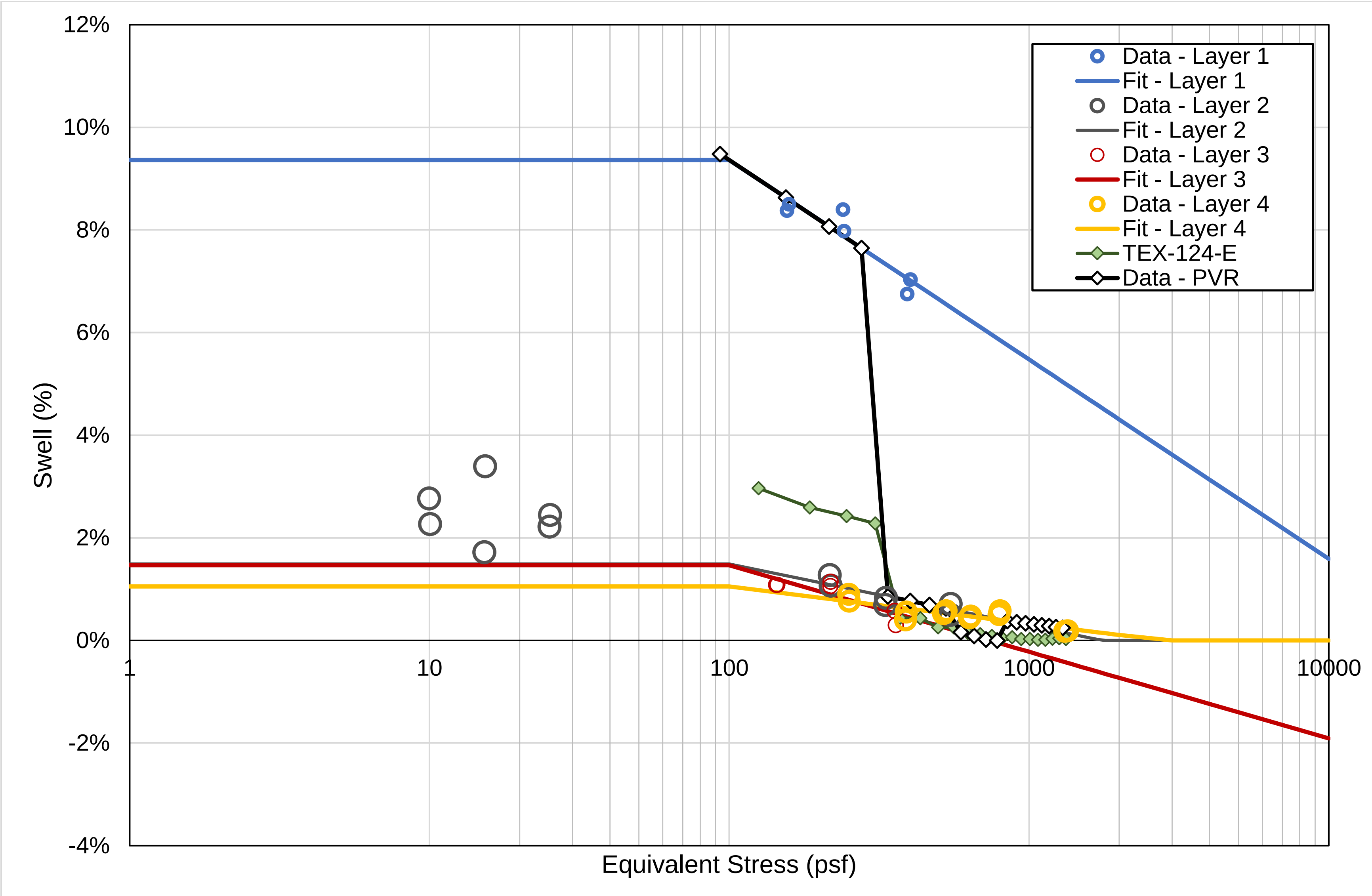
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.37**	**0.74**	
1.0	1	FM972	155	1.80	0.40	
2.0	1	FM972	276	0.80	0.12	
3.0	2	FM 972	402	0.29	0.07	
4.0	2	FM 972	529	0.20	0.04	
5.0	3	FM 972	656	0.15	0.02	
6.0	3	FM 972	783	0.15	0.02	
7.0	4	FM 972	910	0.12	0.01	
8.0	4	FM 972	1038	0.08	0.01	
9.0	4	FM 972	1166	0.05	0.00	
10.0	4	FM 972	1294	0.01	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	FM972	FM 972	FM 972	FM 972	-
	Liquid Limit	57	35	34	40	%
	Plastic Limit	37	19	18	21	%
	Water Content	21%	16%	14%	17%	%
	Dry Unit Weight	100	110	111	109	pcf
	Total Unit Weight	121	127	127	128	pcf
	Thickness	2	2	2	4	ft
	A - Fitting Parameter	-0.039	-0.012	-0.017	-0.007	-
	B - Fitting Parameter	0.1714	0.0389	0.0484	0.0251	-

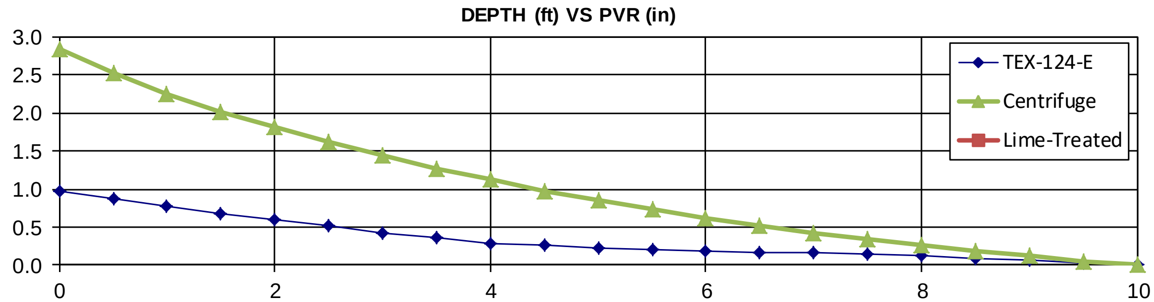
Texas Department of Transportation


Swell (%)
12%
10%
8%
6%
4%
2%
0%
-2%
-4%
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Williamson	
Highway:	FM 972	
CSJ Number:		
District:	Austin	
Boring Number:	2	
Date Sampled:	8/14/2018	
Ground Elevation:		
Station:		
Offset:		



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

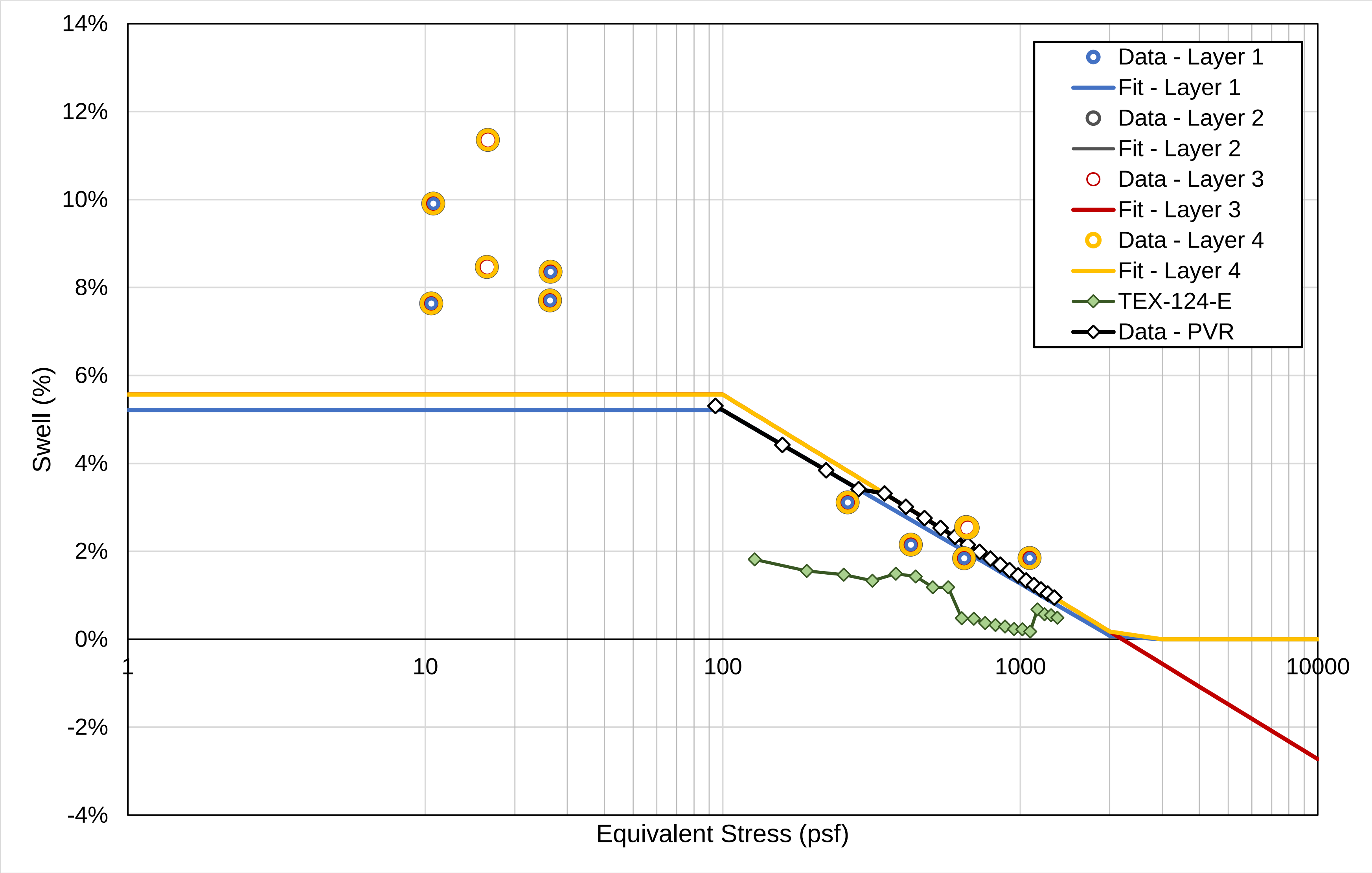
Depth of Treatment
0 ft

SOIL Information

Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	FM 972	FM 972	FM 972	FM 972	-
Liquid Limit	54	54	41	54	%
Plastic Limit	35	35	25	37	%
Water Content	18%	18%	18%	18%	%
Dry Unit Weight	108	108	108	108	pcf
Total Unit Weight	127	127	127	127	pcf
Thickness	2	2	4	2	ft
A - Fitting Parameter	-0.039	-0.041	-0.041	-0.041	-
B - Fitting Parameter	0.1310	0.1386	0.1386	0.1386	-

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.84**	**0.98**	
1.0	1	FM 972	159	2.52	0.78	
2.0	1	FM 972	286	2.02	0.61	
3.0	2	FM 972	413	1.62	0.43	
4.0	2	FM 972	540	1.27	0.29	
5.0	3	FM 972	667	0.98	0.23	
6.0	3	FM 972	794	0.73	0.19	
7.0	3	FM 972	921	0.52	0.16	
8.0	3	FM 972	1047	0.34	0.14	
9.0	4	FM 972	1174	0.19	0.06	
10.0	4	FM 972	1301	0.06	0.00	

Texas Department of Transportation

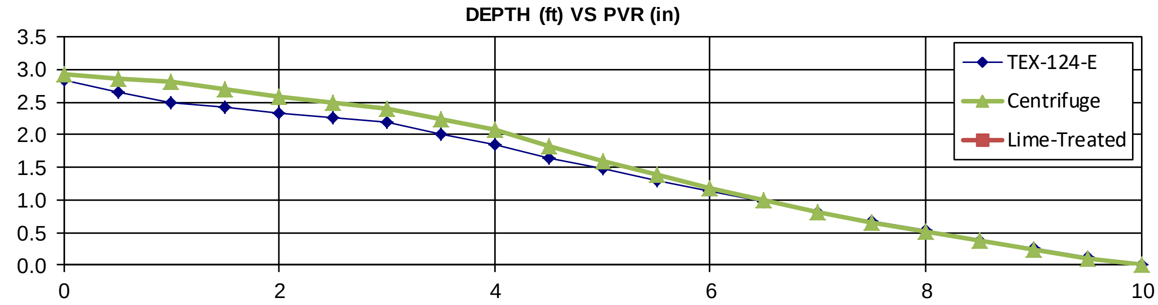


14%
12%
10%
8%
6%
4%
2%
0%
-2%
-4%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

Borings from FM 2563

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information	County:	Bexar
	Highway:	FM 2563
	CSJ Number:	
	District:	San Antonio
	Boring Number:	1
	Date Sampled:	8/14/2017
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, pavement=	0.25	in
	Unit Weight, pavement=	110	pcf
	Height, base=	7	in
	Unit Weight, base=	115	pcf
	Height, top soil=	0	in
	Unit Weight, top soil=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

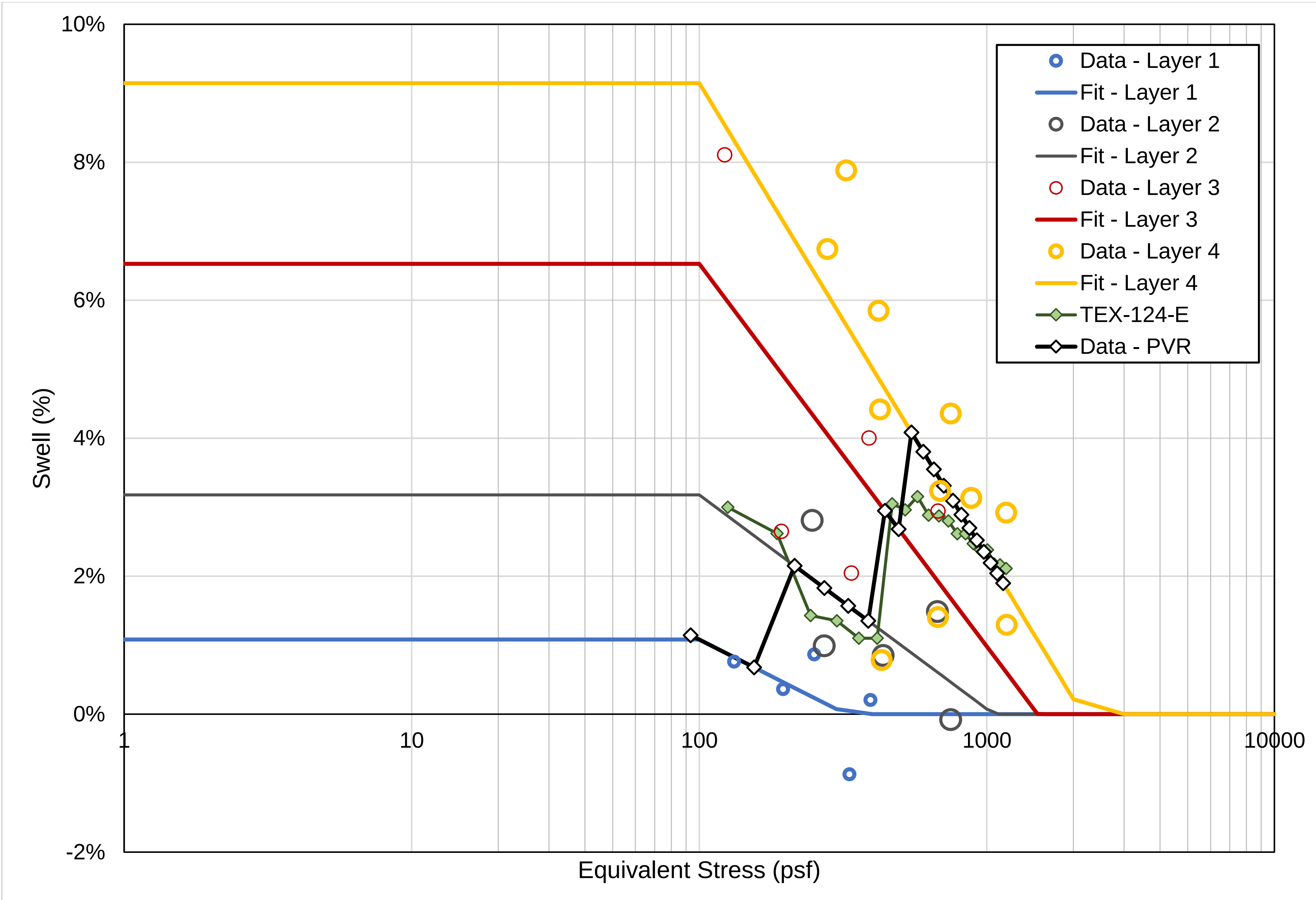
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.93**	**2.83**	
1.0	1	HnB	155	2.86	2.50	
2.0	2	HnB	272	2.69	2.33	
3.0	2	HnB	387	2.48	2.20	
4.0	3	HnB	494	2.23	1.84	
5.0	4	HnB	601	1.82	1.48	
6.0	4	HnB	708	1.38	1.13	
7.0	4	HnB	816	1.00	0.82	
8.0	4	HnB	924	0.66	0.53	
9.0	4	HnB	1031	0.37	0.26	
10.0	4	HnB	1139	0.11	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	HnB	HnB	HnB	HnB	-
	Liquid Limit	60	58	98	96	%
	Plastic Limit	37	36	66	66	%
	Water Content	18%	25%	27%	27%	%
	Dry Unit Weight	103	92	82	85	pcf
	Total Unit Weight	122	115	104	108	pcf
	Thickness	1	2	1	6	ft
	A - Fitting Parameter	-0.021	-0.031	-0.055	-0.069	-
	B - Fitting Parameter	0.0532	0.0939	0.1762	0.2287	-

Texas Department of Transportation

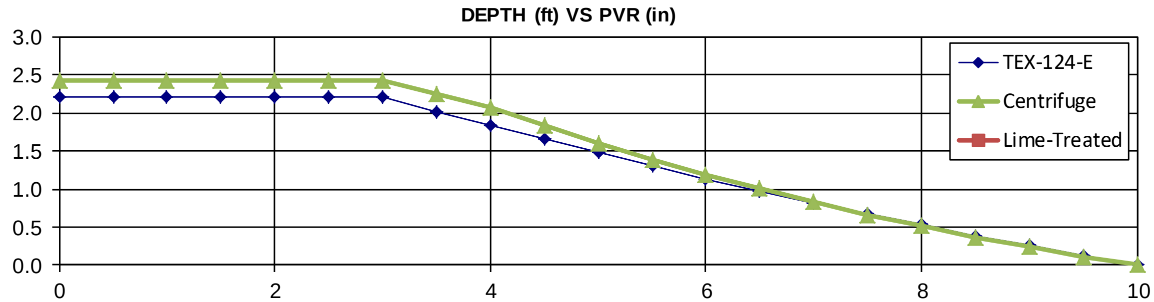


10%
8%
6%
4%
2%
0%
-2%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Bexar
Highway:	FM 2563
CSJ Number:	
District:	San Antonio
Boring Number:	3' REPLACEMENT
Date Sampled:	8/14/2017
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

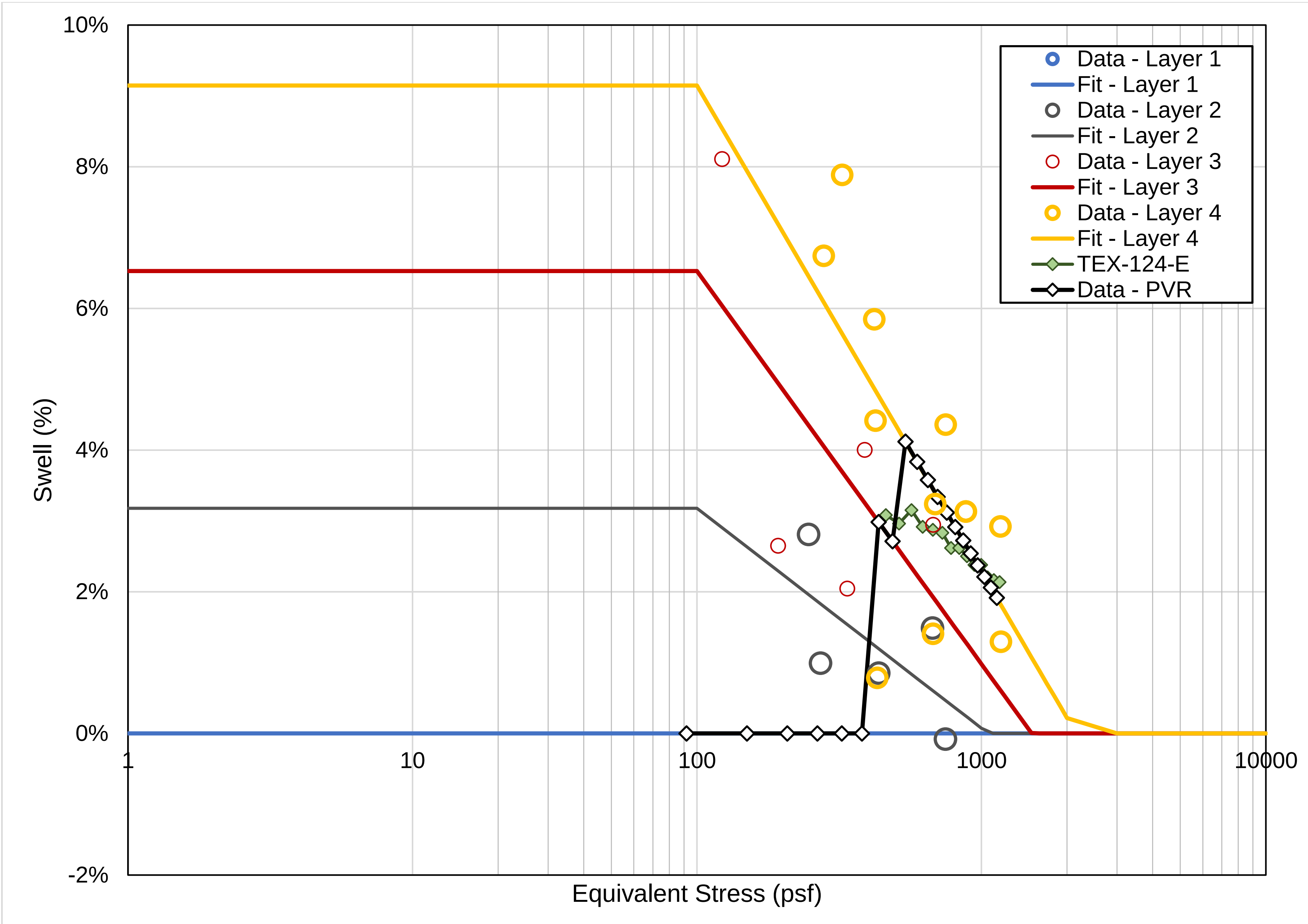
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.42**	**2.21**	
1.0	1	FILL	150	2.42	2.21	
2.0	1	FILL	265	2.42	2.21	
3.0	1	FILL	380	2.42	2.21	
4.0	3	HnB	488	2.25	1.85	
5.0	4	HnB	594	1.84	1.48	
6.0	4	HnB	702	1.39	1.14	
7.0	4	HnB	810	1.00	0.83	
8.0	4	HnB	917	0.67	0.53	
9.0	4	HnB	1025	0.37	0.26	
10.0	4	HnB	1132	0.11	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	FILL	HnB	HnB	HnB	-
Liquid Limit	0	58	98	96	%
Plastic Limit	0	36	66	66	%
Water Content	0%	25%	27%	27%	%
Dry Unit Weight	115	92	82	85	pcf
Total Unit Weight	115	115	104	108	pcf
Thickness	3	0	1	6	ft
A - Fitting Parameter	0.000	-0.031	-0.055	-0.069	-
B - Fitting Parameter	0.0000	0.0939	0.1762	0.2287	-

Texas Department of Transportation

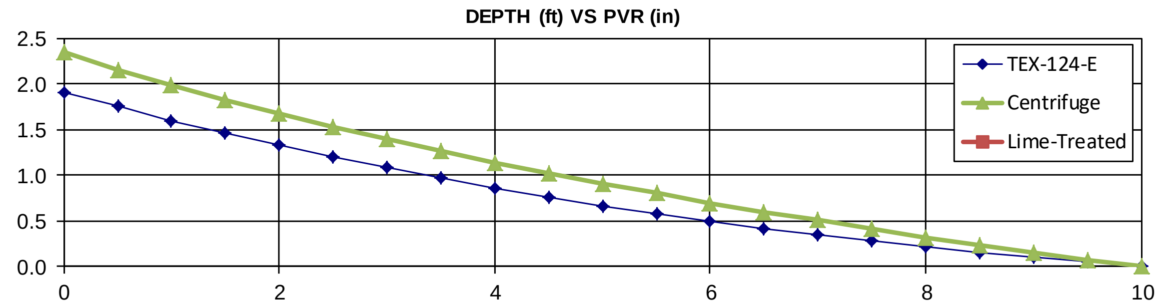


10%
8%
6%
4%
2%
0%
-2%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

Material from US 183

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information	County:	Travis
	Highway:	US-183
	CSJ Number:	
	District:	Austin
	Boring Number:	NB1
	Date Sampled:	6/7/2017
	Ground Elevation:	524
	Station:	
	Offset:	



	Property	Value	Unit
PAVEMENT AND OVERBURDEN Information	Height, $_{pavement}$=	6	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	12	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	170	psf

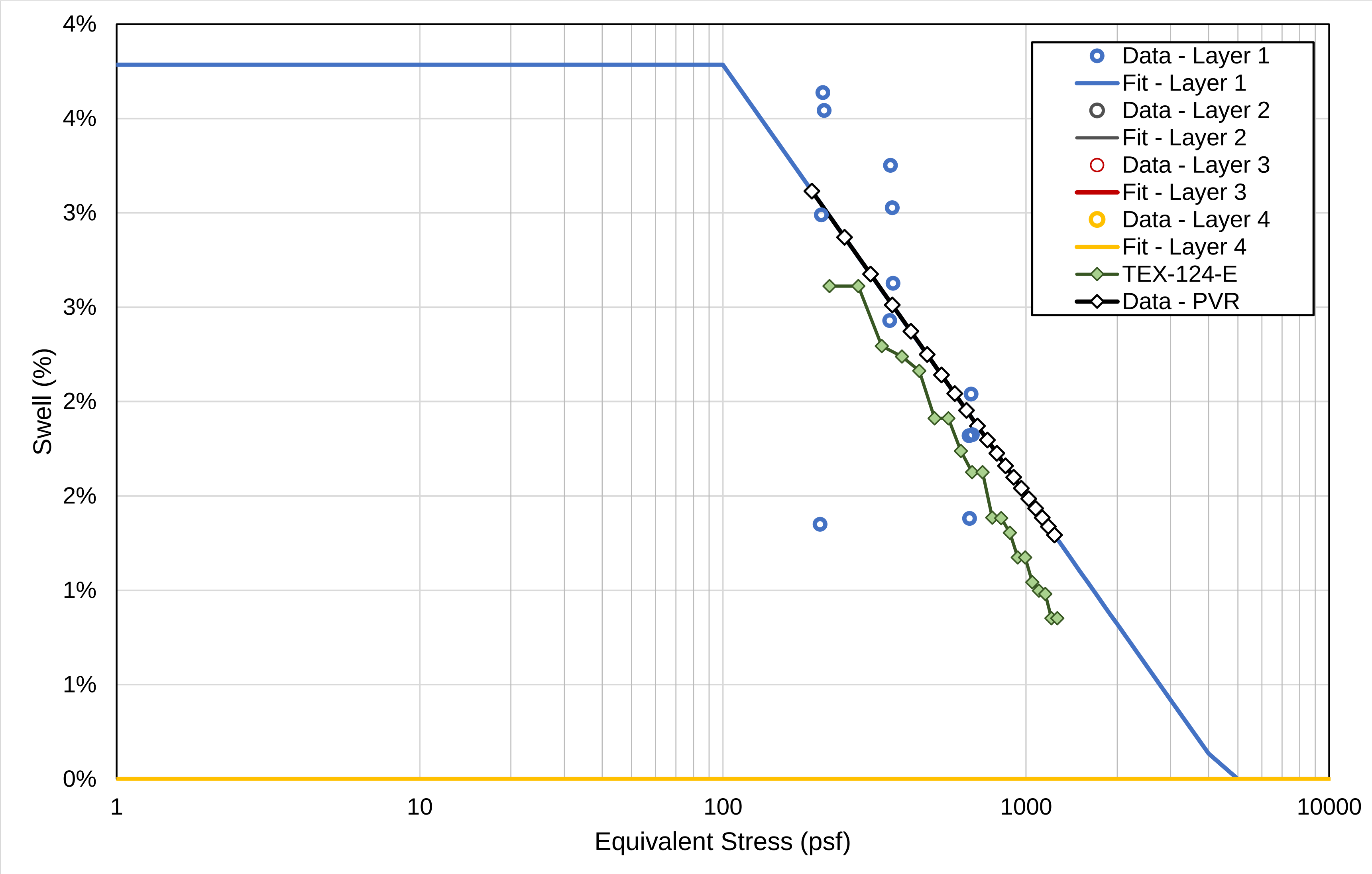
Lime Percentage
0 %

Depth of Treatment
0 ft

	Property	Layer 1	-	-	-	Unit
SOIL Information	Soil:	NB1	-	-	-	-
	Liquid Limit	66	-	-	-	%
	Plastic Limit	47	-	-	-	%
	Water Content	24%	-	-	-	%
	Dry Unit Weight	89	-	-	-	pcf
	Total Unit Weight	110	-	-	-	pcf
	Thickness	10	-	-	-	ft
	A - Fitting Parameter	-0.023	-	-	-	-
	B - Fitting Parameter	0.0834	-	-	-	-

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.34**	**1.91**	
1.0	1	NB1	252	2.16	1.60	
2.0	1	NB1	362	1.82	1.33	
3.0	1	NB1	472	1.53	1.08	
4.0	1	NB1	582	1.27	0.86	
5.0	1	NB1	692	1.03	0.67	
6.0	1	NB1	801	0.81	0.50	
7.0	1	NB1	911	0.60	0.35	
8.0	1	NB1	1021	0.42	0.22	
9.0	1	NB1	1131	0.24	0.10	
10.0	1	NB1	1241	0.08	0.00	

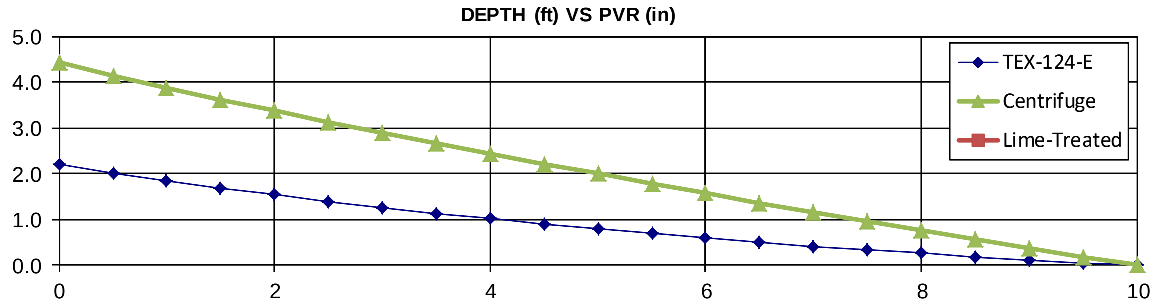
Texas Department of Transportation



4%
4%
3%
3%
2%
2%
1%
1%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Travis
	Highway:	US-183
	CSJ Number:	
	District:	Austin
	Boring Number:	NB2
	Date Sampled:	6/7/2017
	Ground Elevation:	526
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	6	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	12	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	170	psf

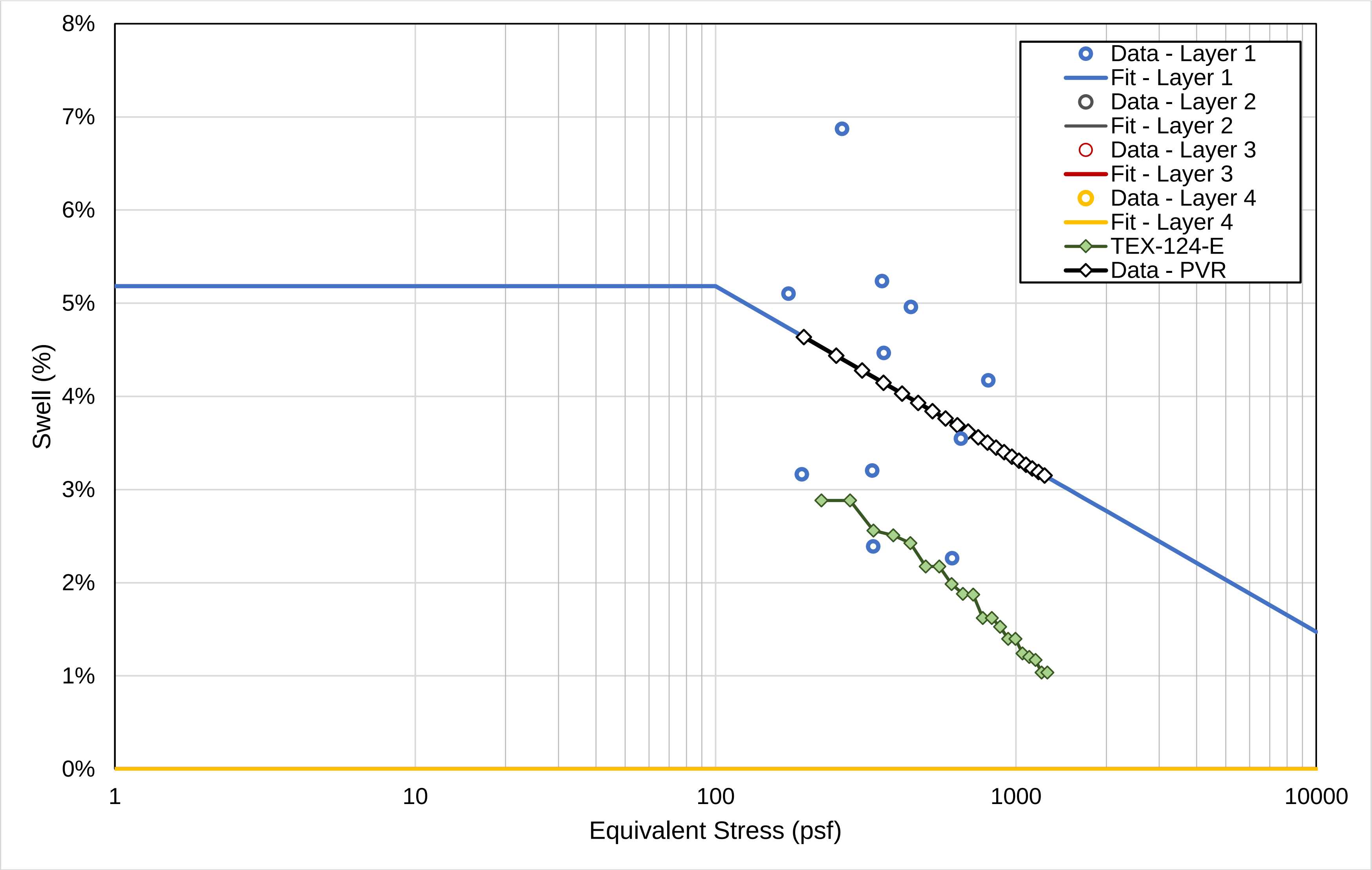
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**4.43**	**2.20**	
1.0	1	NB2	252	4.15	1.85	
2.0	1	NB2	363	3.63	1.55	
3.0	1	NB2	473	3.14	1.27	
4.0	1	NB2	583	2.67	1.02	
5.0	1	NB2	694	2.22	0.80	
6.0	1	NB2	804	1.79	0.60	
7.0	1	NB2	914	1.37	0.42	
8.0	1	NB2	1025	0.97	0.27	
9.0	1	NB2	1135	0.57	0.12	
10.0	1	NB2	1245	0.19	0.00	

SOIL Information	Property	Layer 1	-	-	-	Unit
	Soil:	NB2	-	-	-	-
	Liquid Limit	71	-	-	-	%
	Plastic Limit	51	-	-	-	%
	Water Content	24%	-	-	-	%
	Dry Unit Weight	89	-	-	-	pcf
	Total Unit Weight	110	-	-	-	pcf
	Thickness	10	-	-	-	ft
	A - Fitting Parameter	-0.019	-	-	-	-
	B - Fitting Parameter	0.0889	-	-	-	-

Texas Department of Transportation

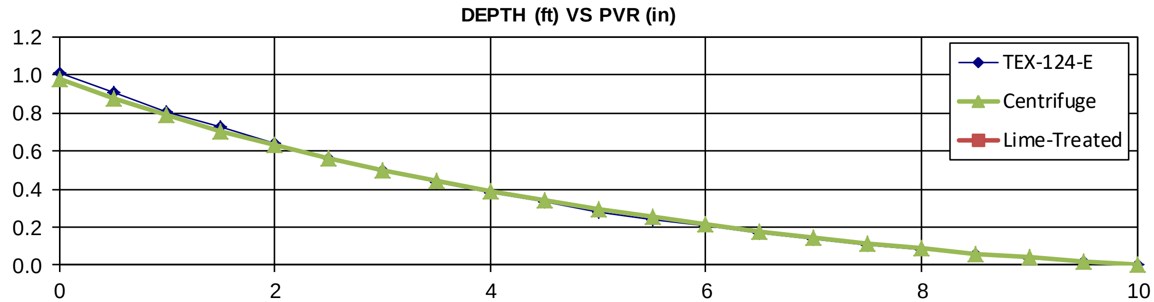


Swell (%)
8%
7%
6%
5%
4%
3%
2%
1%
0%
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Travis
Highway:	US-183
CSJ Number:	
District:	Austin
Boring Number:	NB3
Date Sampled:	6/7/2017
Ground Elevation:	529
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	6	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	12	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	170	psf

Lime Percentage
0 %

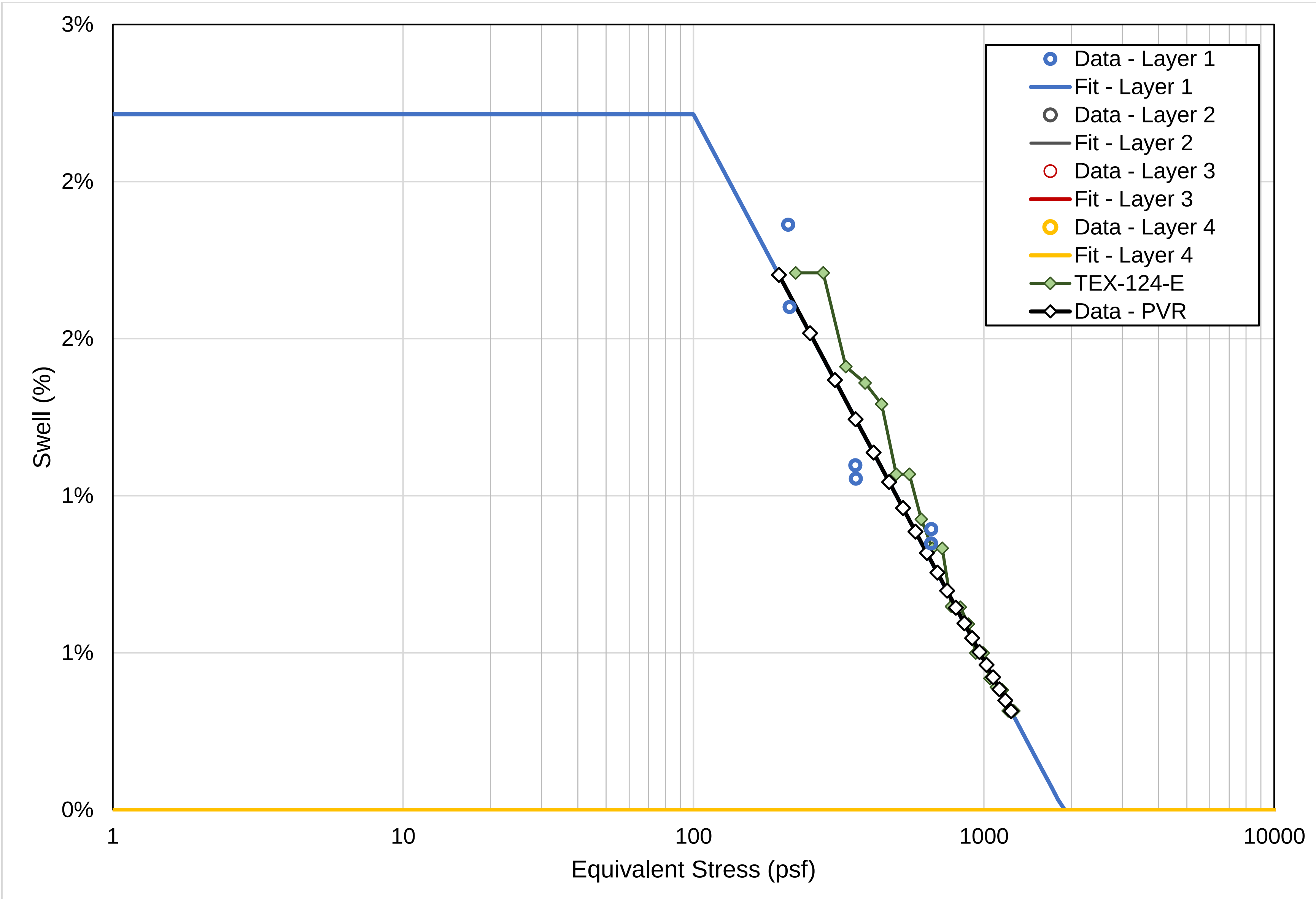
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**0.98**	**1.01**	
1.0	1	NB2	252	0.88	0.81	
2.0	1	NB2	362	0.71	0.64	
3.0	1	NB2	472	0.56	0.50	
4.0	1	NB2	582	0.44	0.38	
5.0	1	NB2	692	0.34	0.28	
6.0	1	NB2	802	0.25	0.20	
7.0	1	NB2	911	0.18	0.14	
8.0	1	NB2	1021	0.12	0.08	
9.0	1	NB2	1131	0.06	0.04	
10.0	1	NB2	1241	0.02	0.00	

SOIL Information

Property	Layer 1	-	-	-	Unit
Soil:	NB2	-	-	-	-
Liquid Limit	58	-	-	-	%
Plastic Limit	41	-	-	-	%
Water Content	23%	-	-	-	%
Dry Unit Weight	89	-	-	-	pcf
Total Unit Weight	110	-	-	-	pcf
Thickness	10	-	-	-	ft
A - Fitting Parameter	-0.017	-	-	-	-
B - Fitting Parameter	0.0569	-	-	-	-

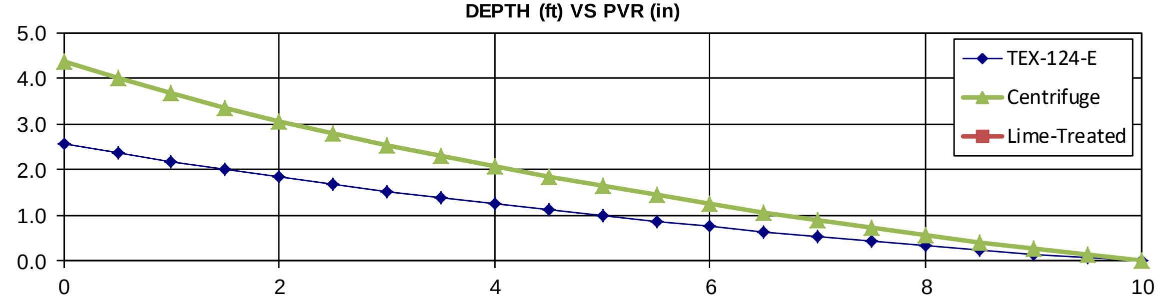
Texas Department of Transportation


3%
2%
2%
1%
1%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Travis
	Highway:	US-183
	CSJ Number:	
	District:	Austin
	Boring Number:	SB1
	Date Sampled:	6/7/2017
	Ground Elevation:	524
	Station:	345+21.281
	Offset:	

DEPTH (ft) VS PVR (in)



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	6	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	12	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	170	psf

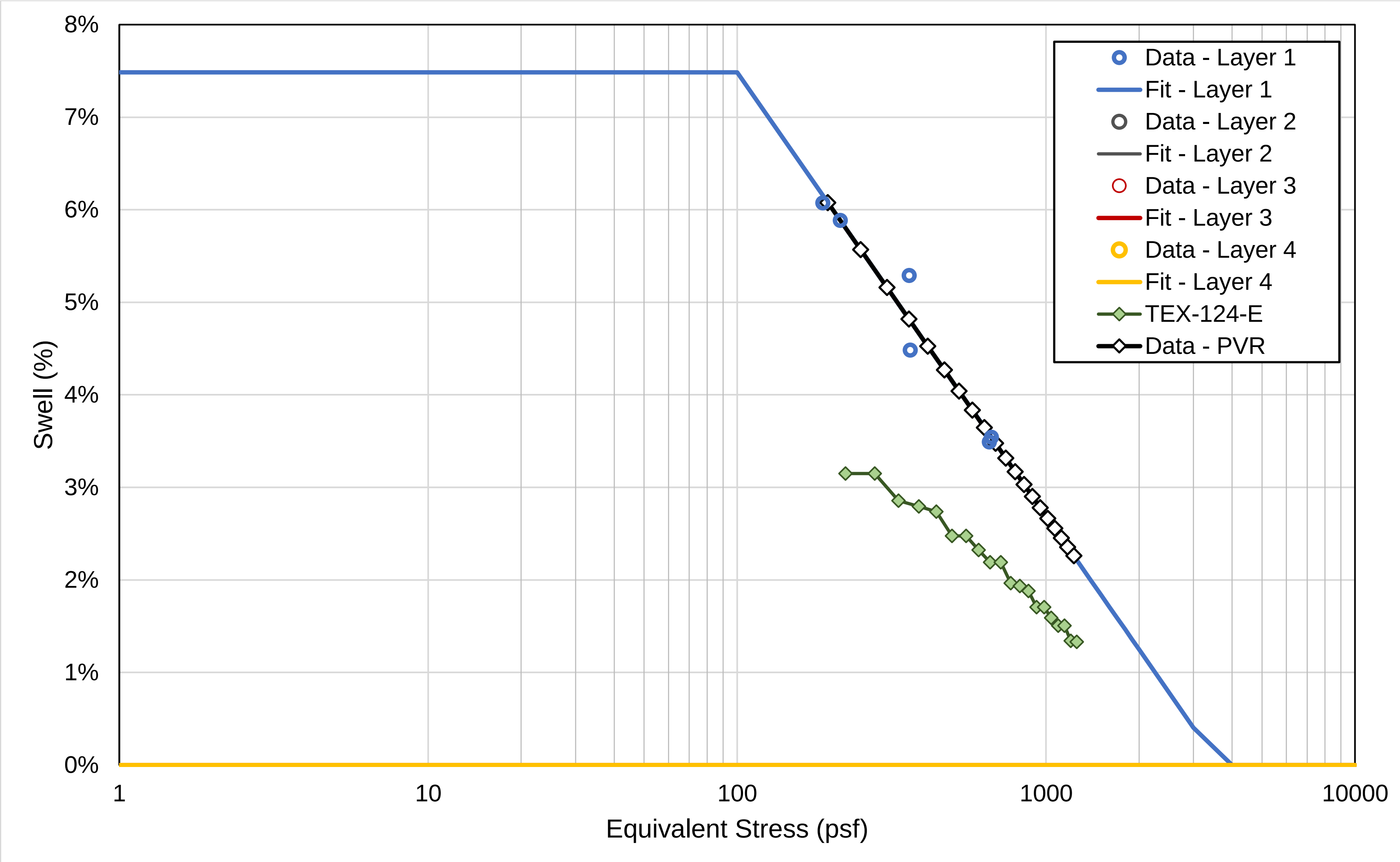
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**4.37**	**2.57**	
1.0	1	SB1	251	4.01	2.19	
2.0	1	SB1	360	3.37	1.85	
3.0	1	SB1	469	2.81	1.54	
4.0	1	SB1	577	2.31	1.25	
5.0	1	SB1	686	1.86	0.99	
6.0	1	SB1	795	1.45	0.75	
7.0	1	SB1	904	1.08	0.54	
8.0	1	SB1	1012	0.74	0.34	
9.0	1	SB1	1121	0.42	0.16	
10.0	1	SB1	1230	0.14	0.00	

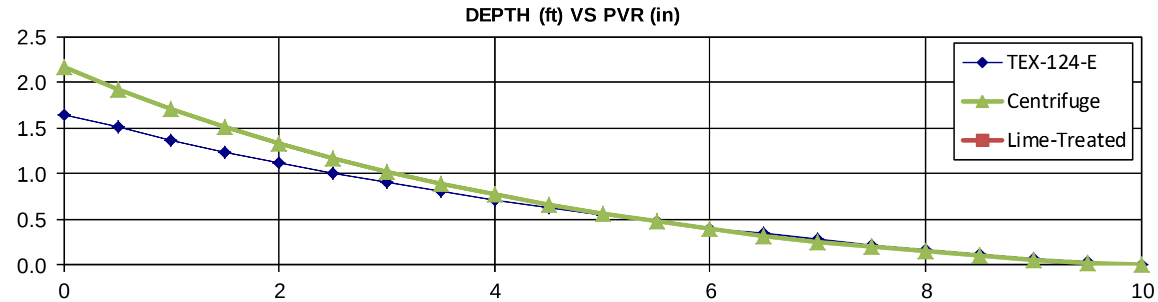
SOIL Information	Property	Layer 1	-	-	-	Unit
	Soil:	SB1	-	-	-	-
	Liquid Limit	76	-	-	-	%
	Plastic Limit	56	-	-	-	%
	Water Content	24%	-	-	-	%
	Dry Unit Weight	87	-	-	-	pcf
	Total Unit Weight	109	-	-	-	pcf
	Thickness	10	-	-	-	ft
	A - Fitting Parameter	-0.048	-	-	-	-
	B - Fitting Parameter	0.1707	-	-	-	-

Texas Department of Transportation


8%
7%
6%
5%
4%
3%
2%
1%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Travis
	Highway:	US-183
	CSJ Number:	
	District:	Austin
	Boring Number:	SB2
	Date Sampled:	6/7/2017
	Ground Elevation:	527
	Station:	343+44.587
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	6	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	12	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	170	psf

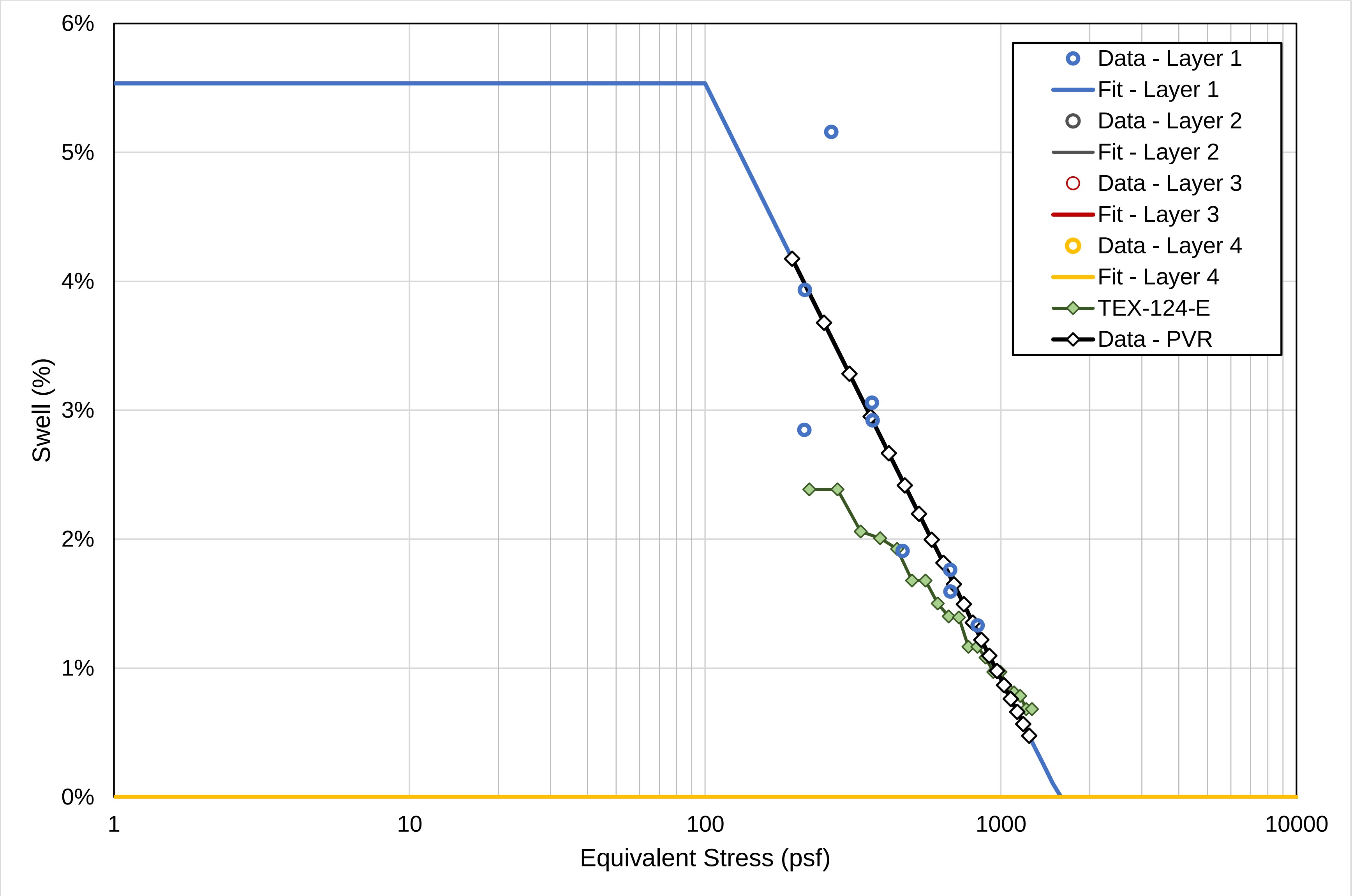
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.18**	**1.65**	
1.0	1	SB2	252	1.93	1.37	
2.0	1	SB2	363	1.51	1.12	
3.0	1	SB2	473	1.17	0.91	
4.0	1	SB2	584	0.90	0.72	
5.0	1	SB2	694	0.67	0.55	
6.0	1	SB2	804	0.48	0.41	
7.0	1	SB2	915	0.32	0.29	
8.0	1	SB2	1025	0.20	0.18	
9.0	1	SB2	1136	0.10	0.08	
10.0	1	SB2	1246	0.03	0.00	

SOIL Information	Property	Layer 1	-	-	-	Unit
	Soil:	SB2	-	-	-	-
	Liquid Limit	61	-	-	-	%
	Plastic Limit	43	-	-	-	%
	Water Content	23%	-	-	-	%
	Dry Unit Weight	90	-	-	-	pcf
	Total Unit Weight	110	-	-	-	pcf
	Thickness	10	-	-	-	ft
	A - Fitting Parameter	-0.046	-	-	-	-
	B - Fitting Parameter	0.1477	-	-	-	-

Texas Department of Transportation

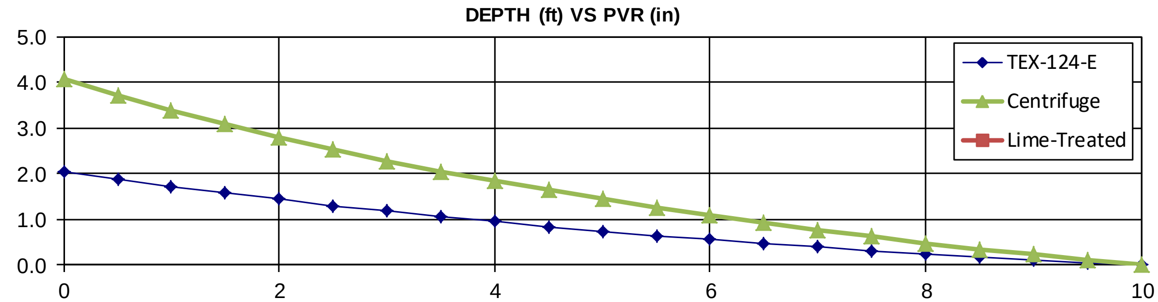


6%
5%
4%
3%
2%
1%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Travis
Highway:	US-183
CSJ Number:	
District:	Austin
Boring Number:	SB3
Date Sampled:	6/7/2017
Ground Elevation:	526
Station:	340+74.592
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	6	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	12	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	170	psf

Lime Percentage
0 %

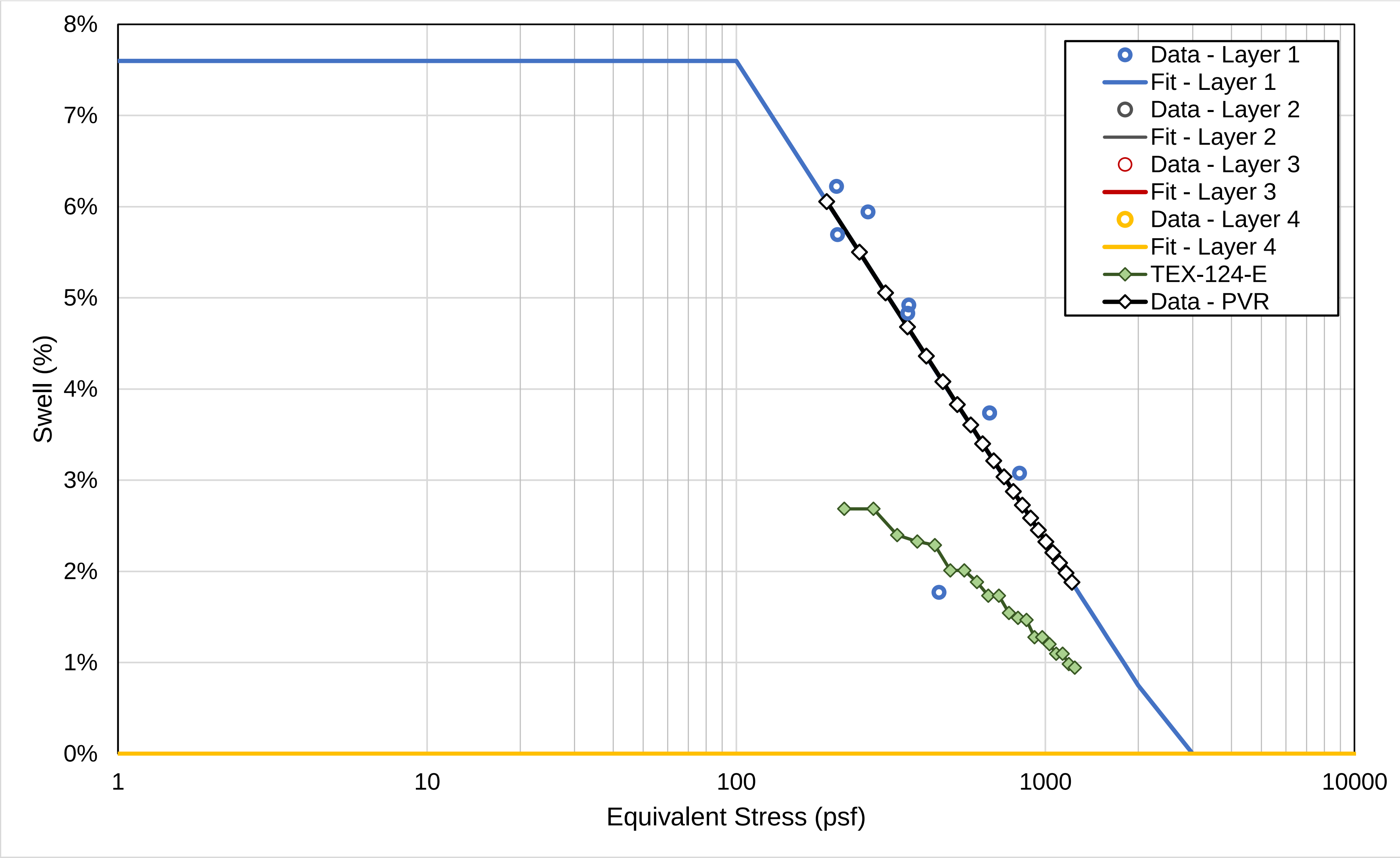
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**4.08**	**2.05**	
1.0	1	SB3	250	3.71	1.73	
2.0	1	SB3	358	3.08	1.44	
3.0	1	SB3	466	2.54	1.18	
4.0	1	SB3	573	2.06	0.95	
5.0	1	SB3	681	1.64	0.74	
6.0	1	SB3	789	1.27	0.56	
7.0	1	SB3	896	0.93	0.40	
8.0	1	SB3	1004	0.63	0.25	
9.0	1	SB3	1112	0.36	0.12	
10.0	1	SB3	1219	0.11	0.00	

SOIL Information

Property	Layer 1	-	-	-	Unit
Soil:	SB3	-	-	-	-
Liquid Limit	72	-	-	-	%
Plastic Limit	50	-	-	-	%
Water Content	23%	-	-	-	%
Dry Unit Weight	88	-	-	-	pcf
Total Unit Weight	108	-	-	-	pcf
Thickness	10	-	-	-	ft
A - Fitting Parameter	-0.053	-	-	-	-
B - Fitting Parameter	0.1813	-	-	-	-

Texas Department of Transportation



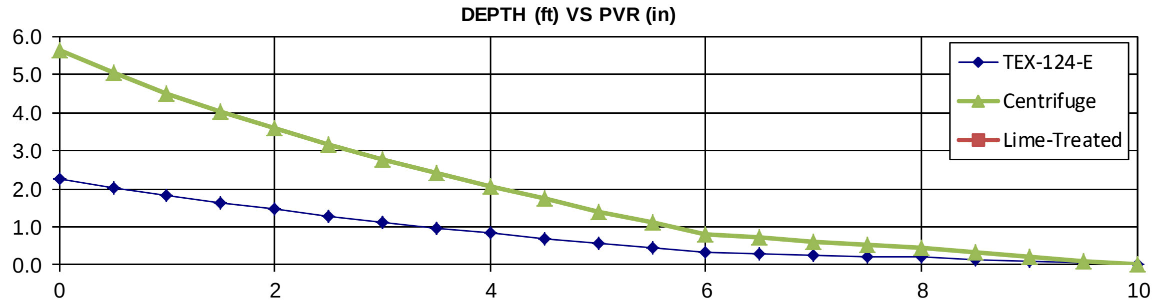
8%
7%
6%
5%
4%
3%
2%
1%
0%
Swell (%)
1
10
100
1000
10000
Equivalent Stress (psf)
Data - Layer 1
Fit - Layer 1
Data - Layer 2
Fit - Layer 2
Data - Layer 3
Fit - Layer 3
Data - Layer 4
Fit - Layer 4
TEX-124-E
Data - PVR

Borings from Turnersville Road

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Travis
Highway:	Turnersville Rd
CSJ Number:	
District:	Austin
Boring Number:	1
Date Sampled:	8/9/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

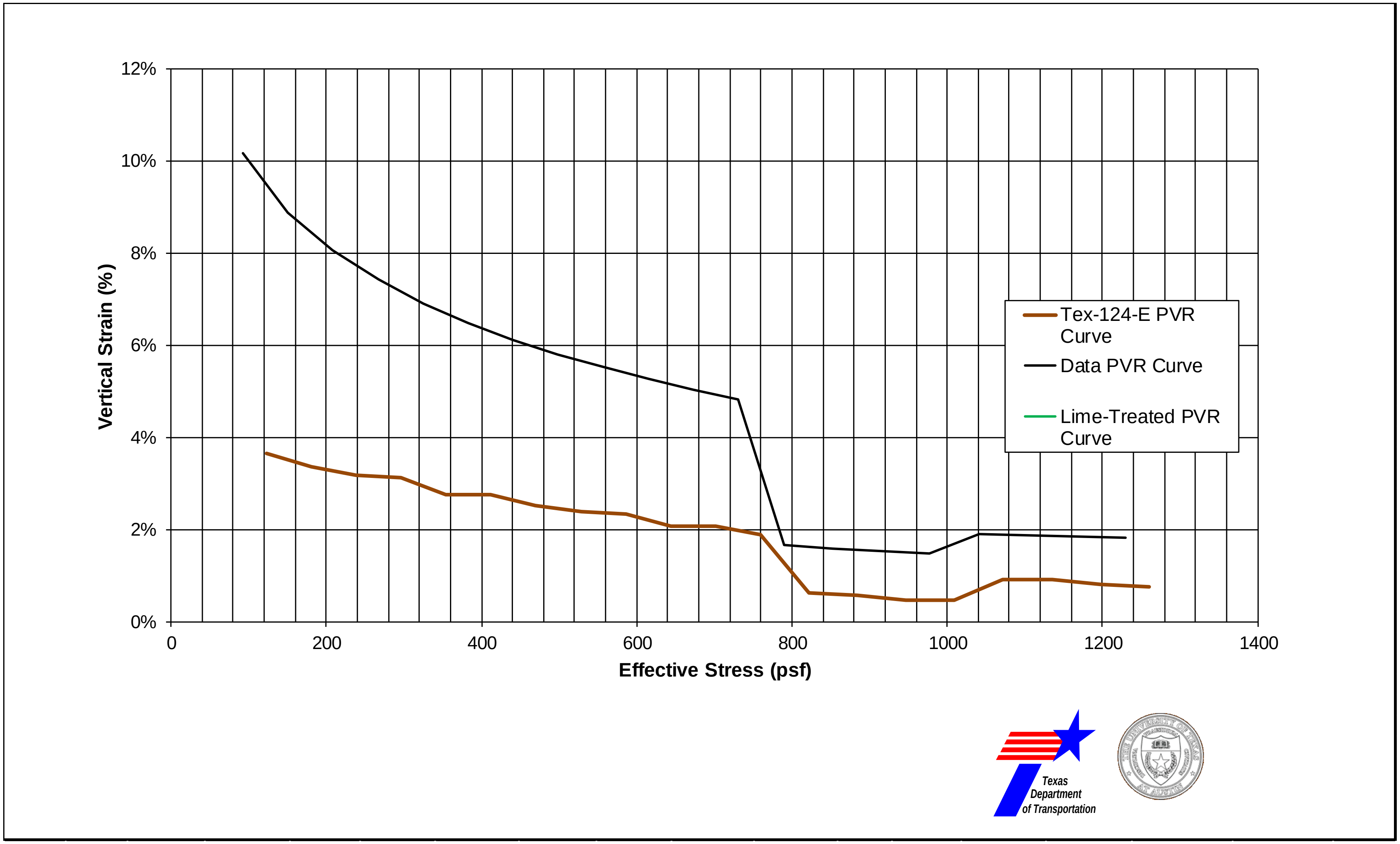
Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**5.65**	**2.26**	
1.0	1	HnB	151	5.05	1.84	
2.0	1	HnB	267	4.03	1.46	
3.0	1	HnB	383	3.17	1.13	
4.0	1	HnB	499	2.41	0.84	
5.0	1	HnB	615	1.73	0.57	
6.0	1	HnB	730	1.11	0.33	
7.0	2	HnB	853	0.72	0.26	
8.0	2	HnB	978	0.54	0.21	
9.0	3	HnB	1103	0.33	0.09	
10.0	3	HnB	1229	0.11	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	-	Unit
Soil:	HnB	HnB	HnB	-	-
Liquid Limit	75	45	58	-	%
Plastic Limit	51	30	41	-	%
Water Content	24%	17%	20%	-	%
Dry Unit Weight	93	106	104	-	pcf
Total Unit Weight	116	125	126	-	pcf
Thickness	6	2	2	-	ft
A - Fitting Parameter	-0.059	-0.020	-0.014	-	-
B - Fitting Parameter	0.2179	0.0736	0.0611	-	-

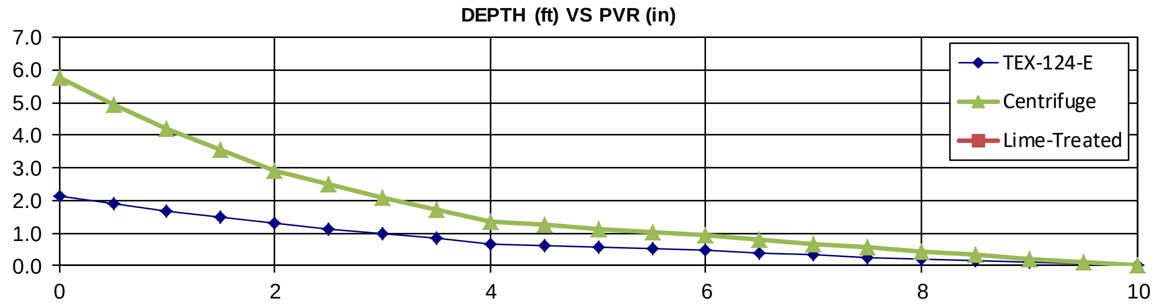
Texas Department of Transportation



12%
10%
8%
6%
4%
2%
0%
Vertical Strain (%)
0
200
400
600
800
1000
1200
1400
Effective Stress (psf)
Tex-124-E PVR Curve
Data PVR Curve
Lime-Treated PVR Curve
Texas Department of Transportation

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Travis
	Highway:	Turnersville Rd
	CSJ Number:	
	District:	Austin
	Boring Number:	2
	Date Sampled:	8/9/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

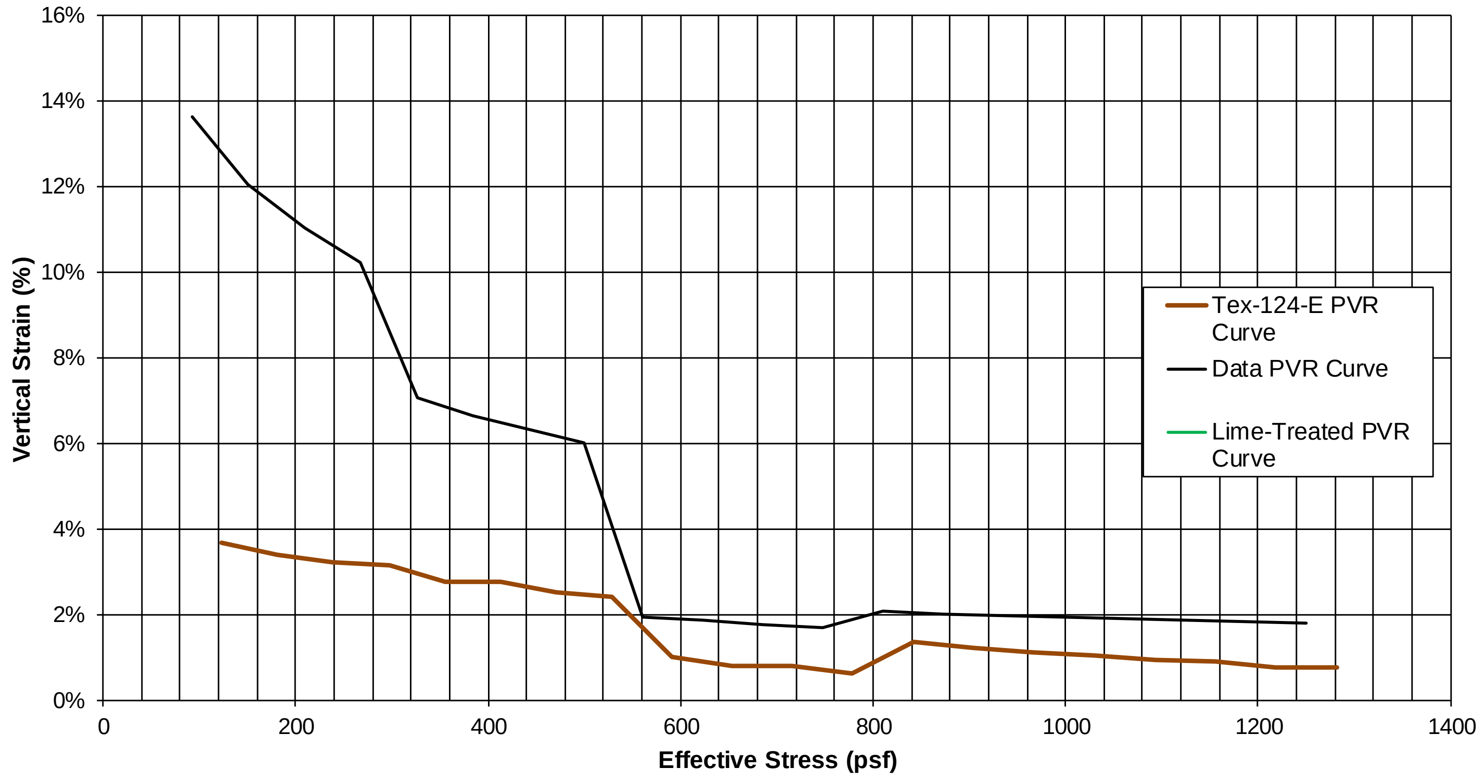
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**5.74**	**2.12**	
1.0	1	HnB	151	4.93	1.69	
2.0	1	HnB	268	3.54	1.31	
3.0	2	HnB	384	2.50	0.98	
4.0	2	HnB	500	1.72	0.68	
5.0	3	HnB	622	1.25	0.57	
6.0	3	HnB	747	1.03	0.49	
7.0	4	HnB	873	0.80	0.33	
8.0	4	HnB	998	0.56	0.20	
9.0	4	HnB	1124	0.33	0.09	
10.0	4	HnB	1250	0.11	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	HnB	HnB	HnB	HnB	-
	Liquid Limit	75	72	46	58	%
	Plastic Limit	51	51	30	41	%
	Water Content	23%	24%	17%	20%	%
	Dry Unit Weight	94	93	106	104	pcf
	Total Unit Weight	116	116	125	126	pcf
	Thickness	2	2	2	4	ft
	A - Fitting Parameter	-0.073	-0.056	-0.020	-0.014	-
	B - Fitting Parameter	0.2799	0.2105	0.0736	0.0611	-

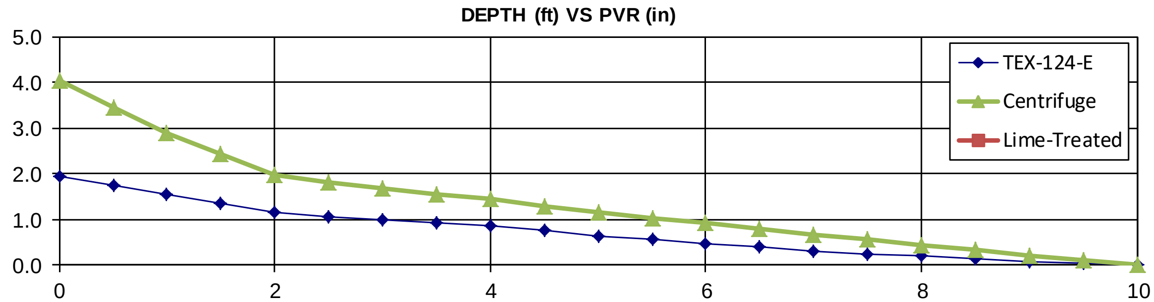
Texas Department of Transportation



16%
14%
12%
10%
8%
6%
4%
2%
0%
Vertical Strain (%)
0
200
400
600
800
1000
1200
1400
Effective Stress (psf)
Tex-124-E PVR Curve
Data PVR Curve
Lime-Treated PVR Curve

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Travis
	Highway:	Turnersville Rd
	CSJ Number:	
	District:	Austin
	Boring Number:	3
	Date Sampled:	8/9/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

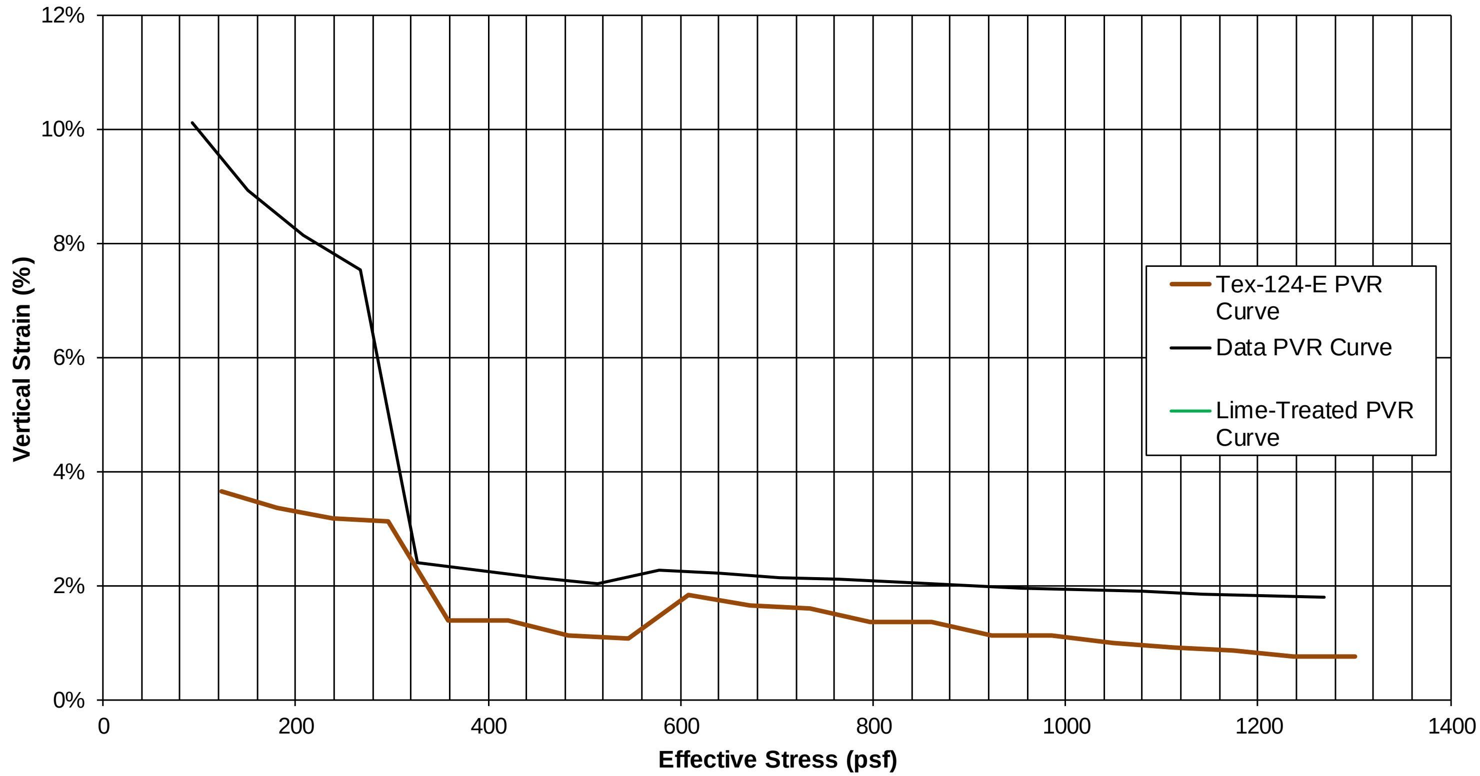
Lime Percentage
0 %

Depth of Treatment
0 ft

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	HnB	HnB	HnB	HnB	-
	Liquid Limit	75	72	46	58	%
	Plastic Limit	51	51	30	41	%
	Water Content	23%	24%	17%	20%	%
	Dry Unit Weight	94	93	106	104	pcf
	Total Unit Weight	116	116	125	126	pcf
	Thickness	0	2	2	6	ft
	A - Fitting Parameter	-0.073	-0.056	-0.020	-0.014	-
	B - Fitting Parameter	0.2799	0.2105	0.0736	0.0611	-

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**4.06**	**1.96**	
1.0	2	HnB	151	3.45	1.54	
2.0	2	HnB	267	2.43	1.16	
3.0	3	HnB	389	1.83	1.00	
4.0	3	HnB	514	1.57	0.86	
5.0	4	HnB	640	1.31	0.65	
6.0	4	HnB	766	1.05	0.48	
7.0	4	HnB	891	0.80	0.33	
8.0	4	HnB	1017	0.56	0.20	
9.0	4	HnB	1143	0.33	0.09	
10.0	4	HnB	1269	0.11	0.00	

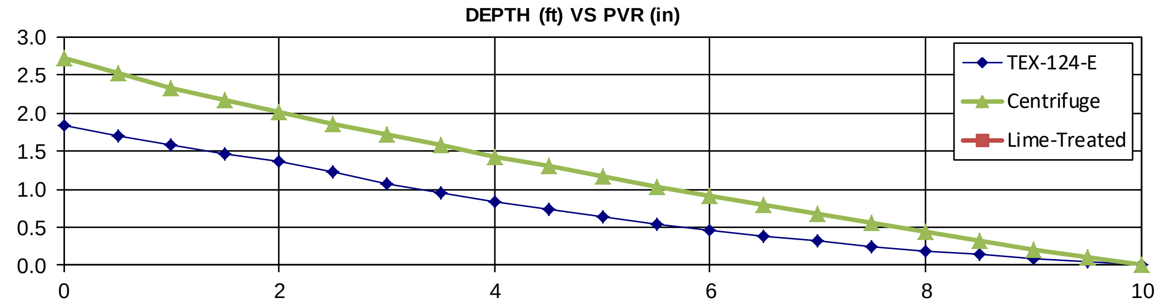
Texas Department of Transportation



12%
10%
8%
6%
4%
2%
0%
Vertical Strain (%)
0
200
400
600
800
1000
1200
1400
Effective Stress (psf)
Tex-124-E PVR Curve
Data PVR Curve
Lime-Treated PVR Curve

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information		
	County:	Travis
	Highway:	Turnersville Rd
	CSJ Number:	
	District:	Austin
	Boring Number:	4
	Date Sampled:	8/9/2018
	Ground Elevation:	
	Station:	
	Offset:	



PAVEMENT AND OVERBURDEN Information	Property	Value	Unit
	Height, $_{pavement}$=	0.25	in
	Unit Weight, $_{pavement}$=	110	pcf
	Height, $_{base}$=	7	in
	Unit Weight, $_{base}$=	115	pcf
	Height, $_{top\ soil}$=	0	in
	Unit Weight, $_{top\ soil}$=	100	pcf
	Surcharge Load =	0	psf
	Total Load =	65	psf

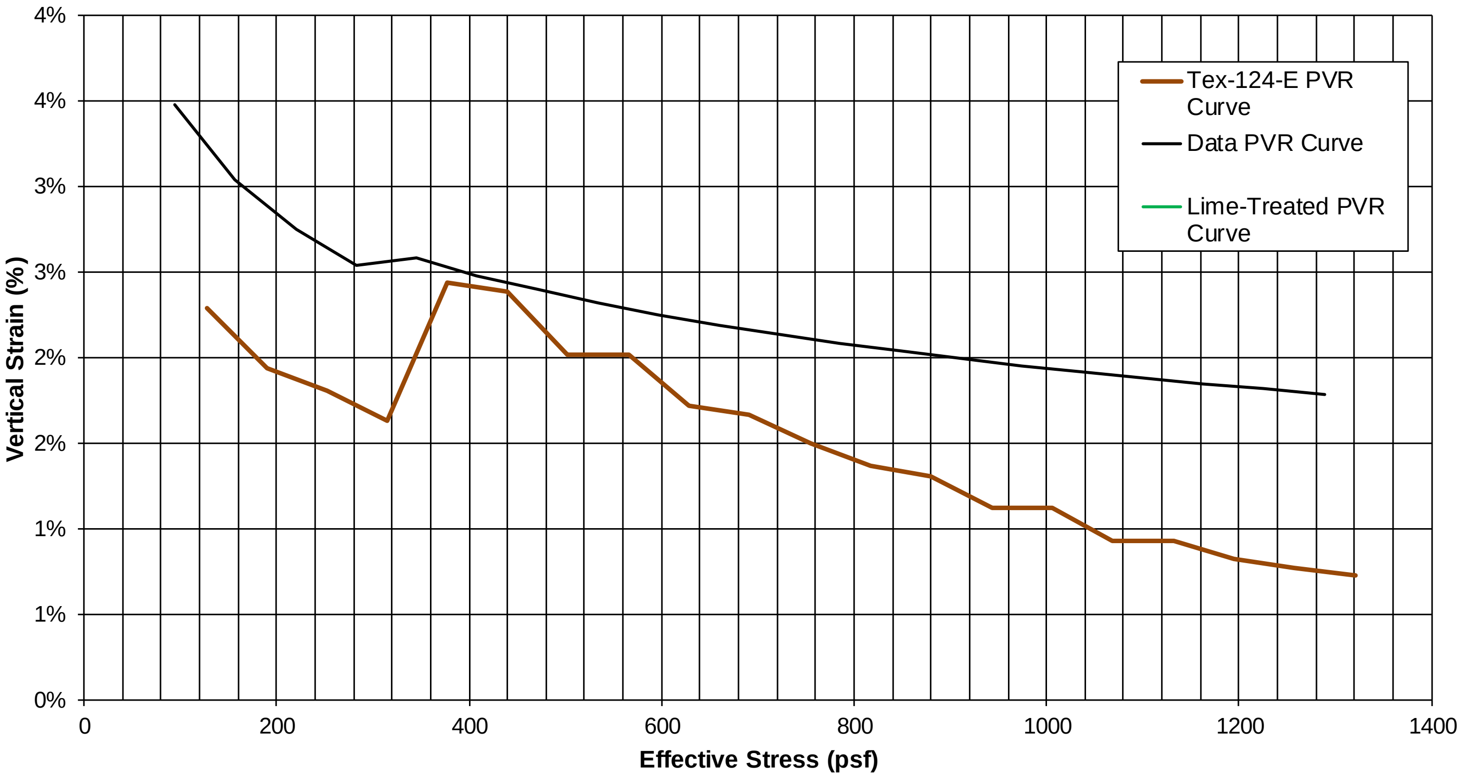
Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.73**	**1.83**	
1.0	3	HnB	157	2.52	1.58	
2.0	3	HnB	282	2.17	1.37	
3.0	4	HnB	408	1.87	1.08	
4.0	4	HnB	534	1.57	0.84	
5.0	4	HnB	660	1.30	0.63	
6.0	4	HnB	786	1.04	0.46	
7.0	4	HnB	911	0.79	0.32	
8.0	4	HnB	1037	0.55	0.19	
9.0	4	HnB	1163	0.33	0.09	
10.0	4	HnB	1289	0.11	0.00	

SOIL Information	Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
	Soil:	HnB	HnB	HnB	HnB	-
	Liquid Limit	75	72	46	58	%
	Plastic Limit	51	51	30	41	%
	Water Content	23%	24%	17%	20%	%
	Dry Unit Weight	94	93	106	104	pcf
	Total Unit Weight	116	116	125	126	pcf
	Thickness	0	0	2	8	ft
	A - Fitting Parameter	-0.073	-0.056	-0.020	-0.014	-
	B - Fitting Parameter	0.2799	0.2105	0.0736	0.0611	-

Texas Department of Transportation

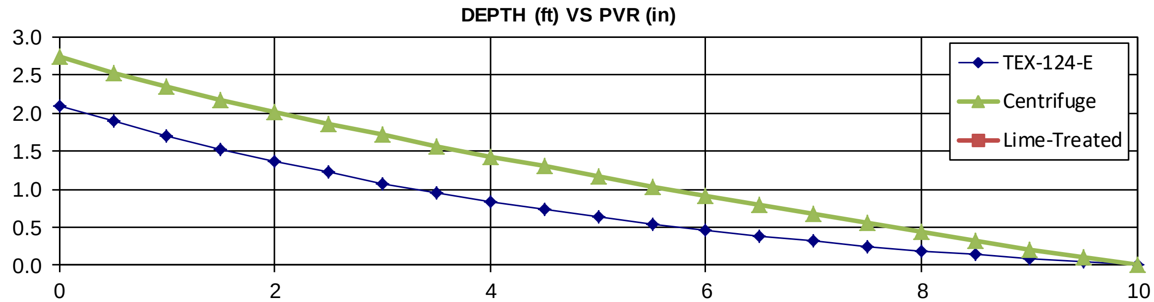


4%
4%
3%
3%
2%
2%
1%
1%
0%
0
200
400
600
800
1000
1200
1400
Vertical Strain (%)
Effective Stress (psf)
Tex-124-E PVR Curve
Data PVR Curve
Lime-Treated PVR Curve

POTENTIAL VERTICAL RISE (PVR)
TEX-124-E And Centrifuge Data

LOCATION Information

County:	Travis
Highway:	Turnersville Rd
CSJ Number:	
District:	Austin
Boring Number:	5
Date Sampled:	8/9/2018
Ground Elevation:	
Station:	
Offset:	



PAVEMENT AND OVERBURDEN Information

Property	Value	Unit
Height, $_{pavement}$=	0.25	in
Unit Weight, $_{pavement}$=	110	pcf
Height, $_{base}$=	7	in
Unit Weight, $_{base}$=	115	pcf
Height, $_{top\ soil}$=	0	in
Unit Weight, $_{top\ soil}$=	100	pcf
Surcharge Load =	0	psf
Total Load =	65	psf

Lime Percentage
0 %

Depth of Treatment
0 ft

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Data PVR [in]	TEX-124-E PVR [in]	Lime-Treated PVR [in]
0	-	-	-	**2.74**	**2.10**	
1.0	4	HnB	158	2.54	1.71	
2.0	4	HnB	284	2.18	1.37	
3.0	4	HnB	410	1.86	1.08	
4.0	4	HnB	536	1.57	0.84	
5.0	4	HnB	662	1.30	0.63	
6.0	4	HnB	788	1.04	0.46	
7.0	4	HnB	913	0.79	0.32	
8.0	4	HnB	1039	0.55	0.19	
9.0	4	HnB	1165	0.33	0.09	
10.0	4	HnB	1291	0.11	0.00	

SOIL Information

Property	Layer 1	Layer 2	Layer 3	Layer 4	Unit
Soil:	HnB	HnB	HnB	HnB	-
Liquid Limit	75	72	46	58	%
Plastic Limit	51	51	30	41	%
Water Content	23%	24%	17%	20%	%
Dry Unit Weight	94	93	106	104	pcf
Total Unit Weight	116	116	125	126	pcf
Thickness	0	0	0	10	ft
A - Fitting Parameter	-0.073	-0.056	-0.020	-0.014	-
B - Fitting Parameter	0.2799	0.2105	0.0736	0.0611	-

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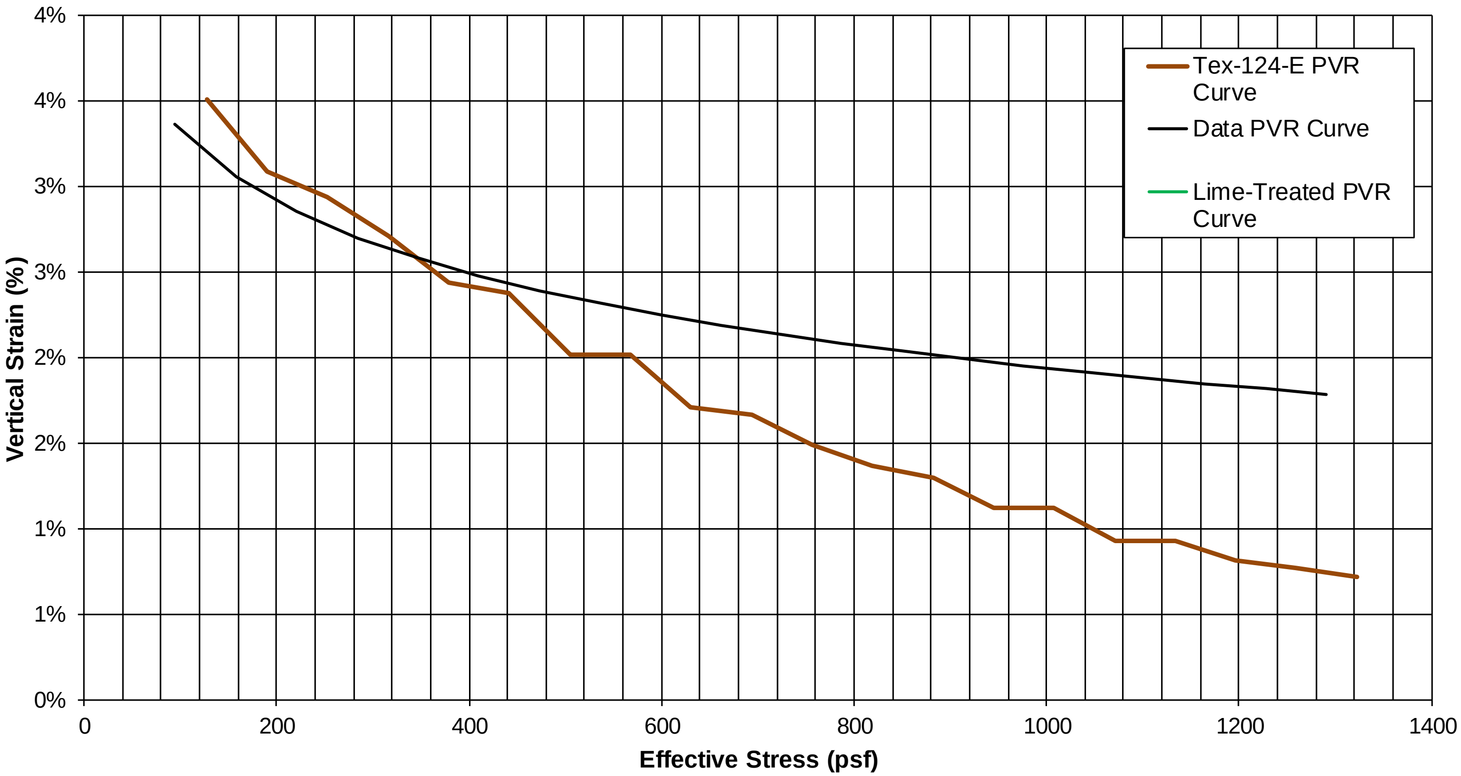

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